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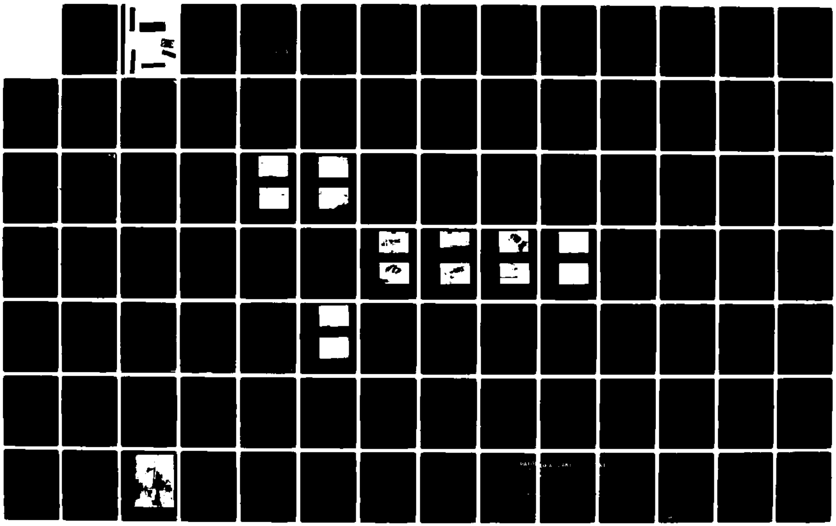

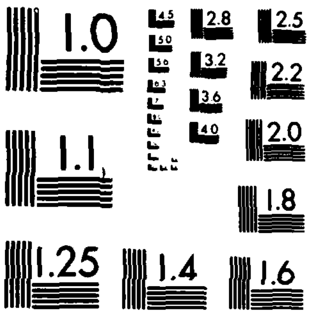

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Dam
Patoka River
Indiana

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

This report covers the construction of the Patoka Dam, Spillway and Dike. Patoka is a flood control structure on the Patoka River, a tributary of the Wabash River in Southwestern Indiana. It is in the flood control structures for the Ohio River Basin. The report contains narratives, charts, photos and construction logs.

IX CHARACTER OF THE FOUNDATION

A. Dam

9-01. The exposed surfaces of the dam foundation were highly variable according to rock types and resistance to weathering. Prior to alluviation of the Patoka Valley during late Pleistocene, the base level of erosion had penetrated through the Beech Creek Limestone leaving high benches of Big Clifty Sandstone. Discussion of the dam foundation character has been outlined according to rock types.

9-02. The Elwren Shale was only exposed at the base of the dam excavation. This rock formed a narrow, relatively flat to slightly undulating surface in the river bed upstream and downstream of the core trench. This flat surface represents the base level of erosion of the Patoka River cutting through the overlying rock types in a very deep (70 ft.) and narrow valley. An exception to the flat lying shale was found in a small monoclinic structure, mentioned in other parts of this report, that separated the two Beech Creek outcrops at the dam centerline. This moncline formed a small (3 ft.) ridge in the center of the excavation and as a result, formed two small bowl-shaped areas upstream and downstream of the dam centerline. The surface of the Elwren Shale was not weathered or grooved by erosion. This rock was impervious and did not transmit groundwater. Bedding in this rock was generally absent and numerous short compaction slickenside gave the surface of this rock a slightly rough texture.

9-03. The next rock in ascending order was the Beech Creek Limestone. With the removal of the valley alluvium, the upper Beech Creek Limestone contact exhibited a major solution zone which extended along the entire lower toe of each abutment. This solutioned horizon basically owed its development to the well developed joint system in the limestone which also served as the principal lateral transfer zone for water percolating down from the overlying rock units. This groundwater system was checked for a long time by burial through valley alluviation. Once exposed again during excavation, this zone produced large quantities of water which basically dewatered both abutments within the dam foundation limits. The solutioned zone in the Beech Creek Limestone was characterized by large cavities which followed the formation outcrop along the base of the excavation on both sides of the valley. Each cavity was partially filled by mineralization deposits formed by the leaching of sulphur and iron through the upper sandstone.

9-04. Where the Beech Creek Limestone was unsolutioned, it was hard, durable, and suitable for construction purposes. Extensive solutioning did not extend a great distance back into the abutments; however open and slightly solutioned joints did extend further. The cutoff trench was constructed to remove these highly solutioned areas. The portion of the cutoff trench on the right abutment extended beyond the limits of extensive solutioning. This side of the trench extended 350 feet back from the original limestone outcrop. At this point, the limestone was highly jointed with occasional small solutioned areas along joint planes. The large solutioned areas and joint faces were not filled with the Terra Rosa clays found in the upper limestones. In fact, most solutioning resulted in voids, some partially filled by mineralization. The cutoff trench on the left abutment was cut approximately 150 feet back of the original outcrop without reaching the end of the extensive solutioning. However, grouting indicated that this highly solutioned zone did not extend more than 50 feet beyond the trench.

9-06. The next rock in ascending order was the Big Clifty Sandstone. This rock was highly resistant to weathering and formed steep side slopes, slightly greater than 1 vertical to 1 horizontal, on both abutments. Small benches were formed at the top of this rock and near the bottom on the right abutment as mentioned in the above paragraph. The benches at the top were approximately 150 feet wide on the left abutment and approximately 20 feet on the right abutment. A wider bench upstream near the tower provided the foundation for the Stage I cofferdam. The thin bedding and friable nature of this rock caused minutely irregular slope surfaces which resulted in small overhangs requiring chipping and packing. This also resulted in a minute stair-stepped surface on both abutments. The rock was generally soft and poorly cemented. Several thin uncemented sand seams were noted on the left abutment, but not on the right abutment. These soft seams were discontinuous laterally into the abutment as noted during the grouting operations. This sandstone was generally porous and allowed the transmission of groundwater to the lower limestone. After excavation, the surfaces of this rock remained dry with no seepage.

9-07. Next in ascending order was the shale member of the Golconda Formation. This rock was thin bedded and soft causing some problems protecting it from equipment damage and weathering prior to embankment operations. This rock was impervious and consequently did not create seepage problems during construction. This rock was worked during the outlet works contract and was left exposed between the tower and dam construction. As a result, problems occurred during dam construction on the right abutment. A small cutoff trench was required for removal of the affected shale near the conduit. The foundation character of this shale was typical of soft, thin bedded compaction shales.

9-08. The limestone member of the Golconda Formation was the next rock in ascending order found in the dam foundation. This rock was characterized by extensive solutioning along joints and bedding planes. Solutioning was noted in all portions of the dam foundation, but diminished slightly in back of the abutments. Most solutioned areas were filled with Terra Rosa clays of depositional and residual origins. These clays were generally saturated, weak, and unstable when first exposed. Upon drying, however, shear strengths increased as noted on the right abutment where the solutioned areas were uncovered during outlet construction and treated during dam construction. The limestone was highly jointed and broke along joints and bedding planes to form large blocks. Weathering depths varied depending upon the extent of solutioning. The limestone itself proved to be hard and durable, and was utilized in the upstream riprap portion of the dam. Where originally uncovered, vertical slopes were prevalent with much dissection and alterations caused by the high degree of solutioning and weathering. Some groundwater was transmitted through the limestone along joints and bedding planes, but most of the water was blocked by the interstitial clays. During construction, large cavities and overhangs were noted in areas where the clay filling was removed. These areas required extensive dental treatment. This limestone was not uncovered in other areas of the foundation, but was encountered during grouting operations on the right bank of the dam and between the dam and the spillway.

9-09. Next in ascending order were the shales, indurated clays and sandstones of the Hardensburg Formation. These rocks were extremely soft and poorly cemented, causing problems during construction. In addition, extensive weathering had weakened these rocks. On the right abutment, this formation was exposed between outlet works and dam construction. Grouting in this exposed portion of the Hardensburg Formation proved difficult and resulted in required removal of a portion of this rock that could not be grouted satisfactorily. Except for the 6-foot sandstone bed at the top of the right abutment, and a 2-4 foot sandstone bed at the base of the Glen Dean Limestone on the left abutment, most of the formation proved to be indurated clay. This formation was basically impervious. The soft condition of the Hardensburg Formation apparently existed since deposition as evidenced by several thin vertical sandstone dikes found within the shale during final cleanup. These two dikes were observed on the right abutment and these two dikes were 0.05 and 0.5 foot thick. The shales were generally flat lying and very poorly bedded. Finished grades were generally smooth and soft. Final surfaces were air cleaned and wetted immediately before impervious embankment placement.

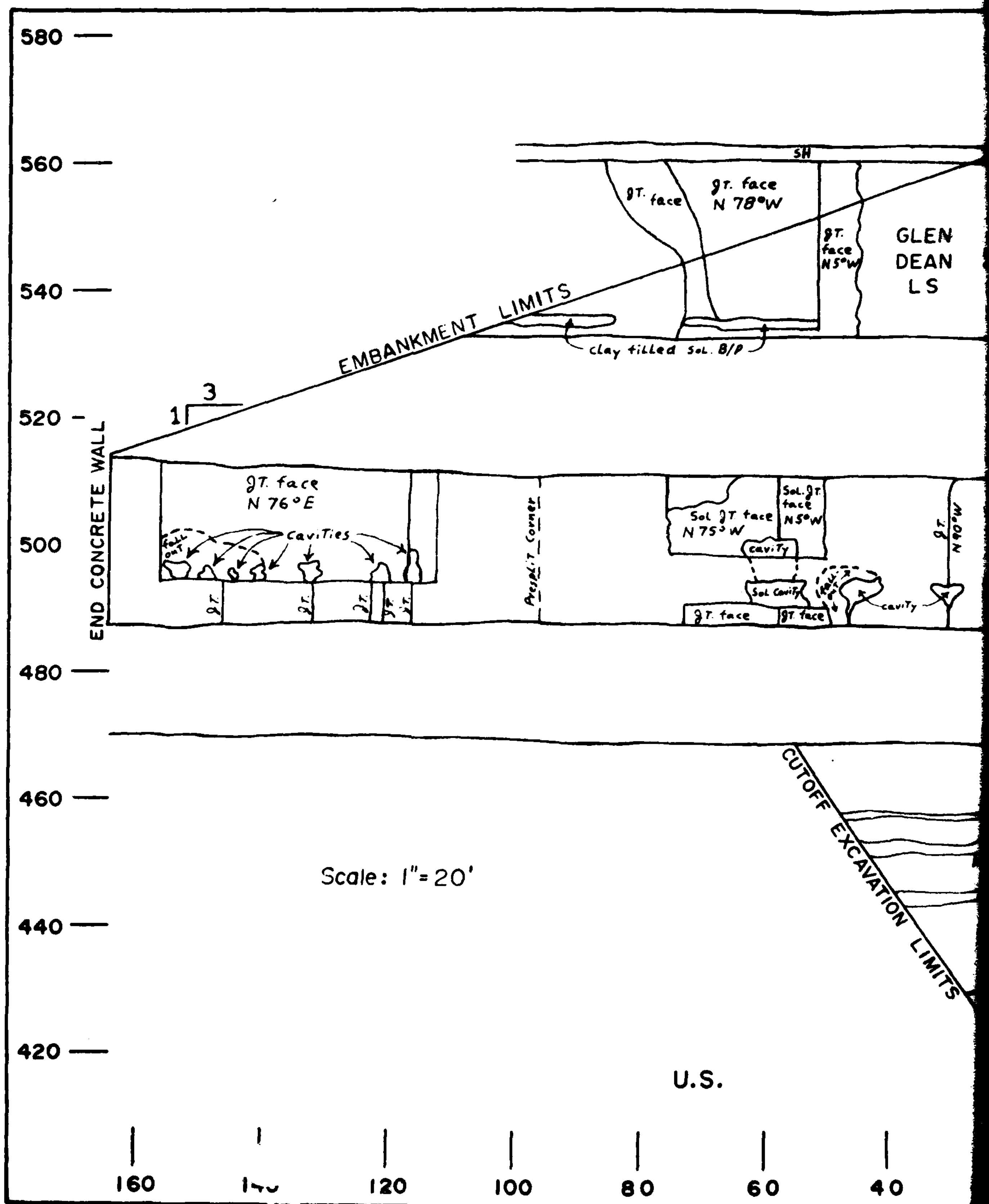
9-10. On the left abutment of the dam, the Hardensburg Formation was found to be in better condition, due mostly to the reduced exposure time during construction. Grouting methods were altered in this area to treat the soft rock. Still, a few breakouts were noted during grouting of reversed angle holes. On the left abutment, a large bench at the top of the Hardensburg Formation was formed by additional removal of the higher Glen Dean Limestone. Beyond the bench, a steep 1V to 1H slope was exposed through the Hardensburg Shale to the top of the lower Golconda Limestone. Although soft, neither the flat bench nor the steep slope weathered appreciably during exposure and neither caused problems during construction. However, a 2-foot thick bed of sandstone at the top of the Hardensburg bench on the left abutment required removal due to ungrouted joints, cracks and fissures.

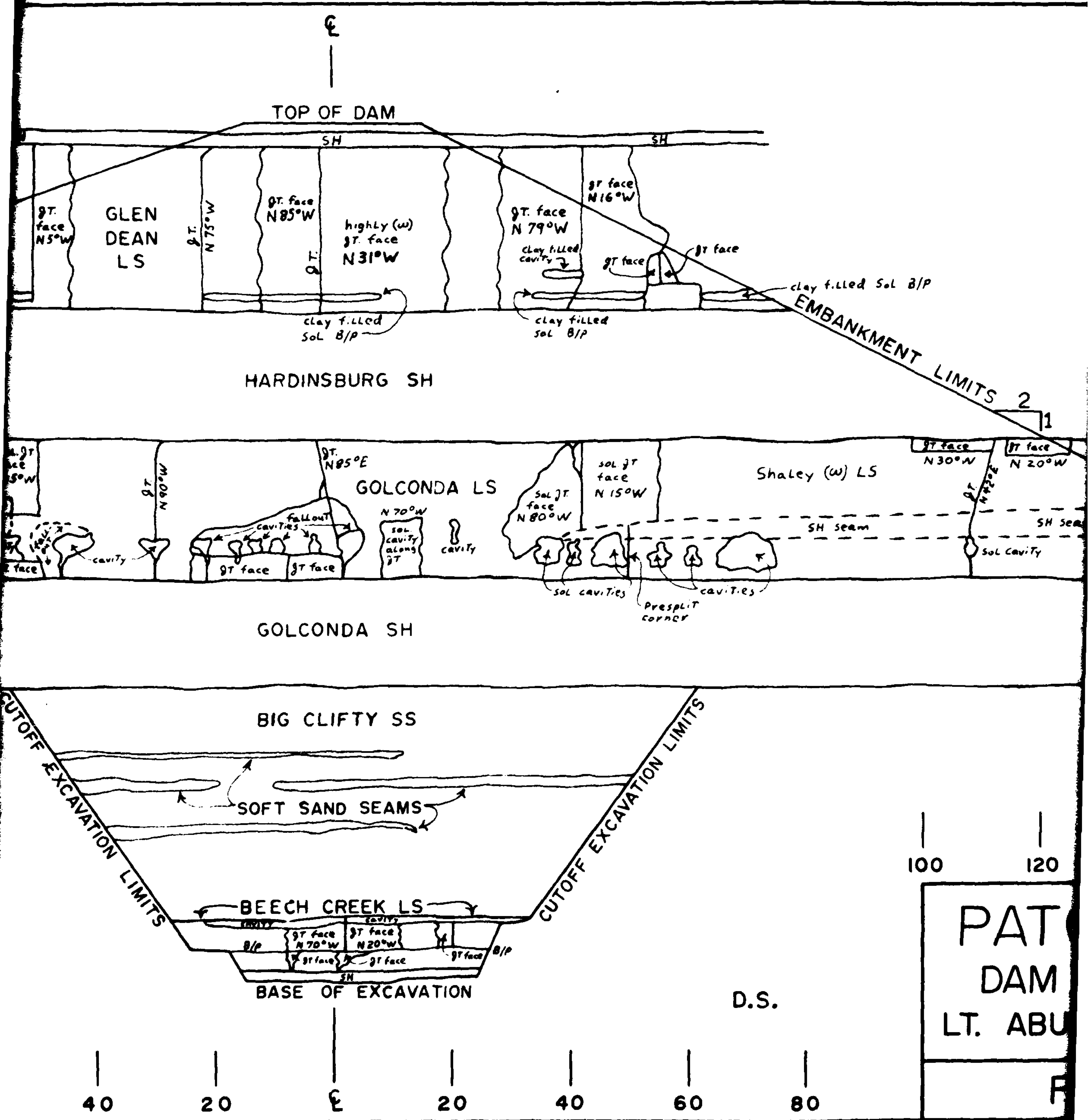
9-11. The next rock in ascending order was the Glen Dean Limestone. The final exposed face of this rock on the left abutment was relatively sound, but was still characterized by solutioning along joints and bedding planes. Explicitly, one vertical joint near the centerline and striking transversely across the backwall was extensively solutioned. This joint formed a portion of the backwall of the final excavation. A second highly solutioned vertical joint was encountered at the upstream corner of the backwall. However, this second joint did not exert as much influence on the foundation treatment as the one near the centerline. A third solutioned vertical joint was encountered on the downstream corner of the backwall, but was less solutioned than the other two joints. In addition to the vertical solutioned joints, a prominent bedding plane approximately 3 feet above the base of the limestone was highly solutioned. The vertical joint near the dam centerline and the solutioned bedding plane were filled to some extent with clay. Numerous open and stained joints and cracks were observed on the back slope of the exposed limestone creating an irregular surface, especially along the upper portion of the limestone. The upper shaley one-half of the limestone was also affected by weathering and staining to a higher degree than the lower one-half.

9-12. The final excavation in this limestone left two wingwalls on the upstream and downstream left abutment limits. Both these wingwalls were solutioned and weathered to a much higher degree than the center portion of the cut. Specifically, both inside corners of the wingwalls at the backwall were extremely solutioned. As a result of solutioning, both wingwalls presented highly rough and irregular surfaces with interstitial clay seams. The intact portion of the Glen Dean Limestone proved to be hard and durable. As a consequence, this rock was utilized in the upstream slope protection. The bedding and joint patterns created a blocky structure to the excavated rock. Generally, the upper portion of the limestone was shaley, while the lower portion was not. In particular, a thick shaley zone in the middle of the limestone separated the limestsone into distinct parts. As noted previously, the Glen Dean Limestone was absent in the foundation portion of the right abutment.

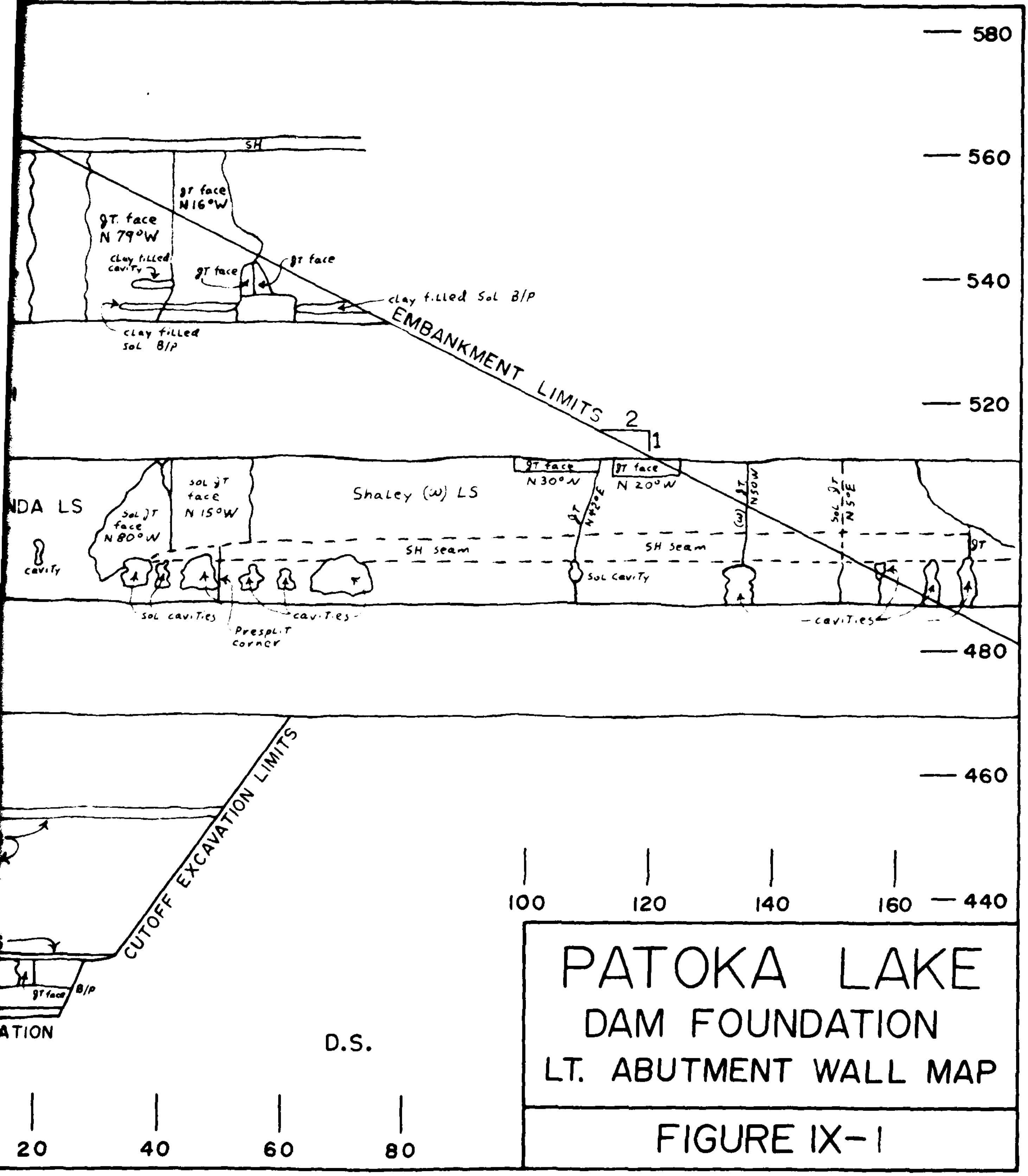
9-13. A thin 3-foot shale seam at the top of the Glen Dean Limestone on the left abutment was all that remained after erosion of the thick interbedded shale and limestone members of the upper Glen Dean Formation. In the center portion of the final backslope, this thin shale was in good condition and generally unweathered. Towards both ends of the face, weathering was more evident along bedding planes and fractures. A gradational contact with the limestone was noted. The upper contact with the overlying sandstone was abrupt and undulating. Little weathering was noted at the upper contact. This shale was impervious, slaked slightly upon exposure, was poorly cemented, and was soft to moderately hard. This shale has been exposed following construction and in that time has formed a slight overhang of the above sandstone.

9-14. The Mansfield Formation was encountered on the left abutment portion of the dam, but was found above elevation 564. Consequently, this rock had little effect upon the dam foundation. Poor exposures were observed at the dam proper. This rock was encountered in other portions of the foundation and will be discussed in the respective portions of this section. Where exposed above the left abutment, this rock was poorly cemented and subjected to freeze and thaw damage. As a result, some spalling of this exposed rock has been experienced subsequent to construction.


580
560
540
520
500
480
460
440
420
SH
JT. face
JT. face
N 78°W
JT. face N5°W
GLEN
DEAN
LS
EMBANKMENT LIMITS
clay filled Sol. B/P
3
1
END CONCRETE WALL
JT. face
N 76°E
Fall out
Cavities
JT.
JT.
JT.
JT.
JT.
Presplit Corner
Sol JT face
N 75°W
Sol. JT. face N5°W
cavity
Sol Cavity
JT. face
JT. face
cavity
JT. N40°W
CUTOFF EXCAVATION LIMITS
Scale: 1"=20'
U.S.
160
120
100
80
60
40



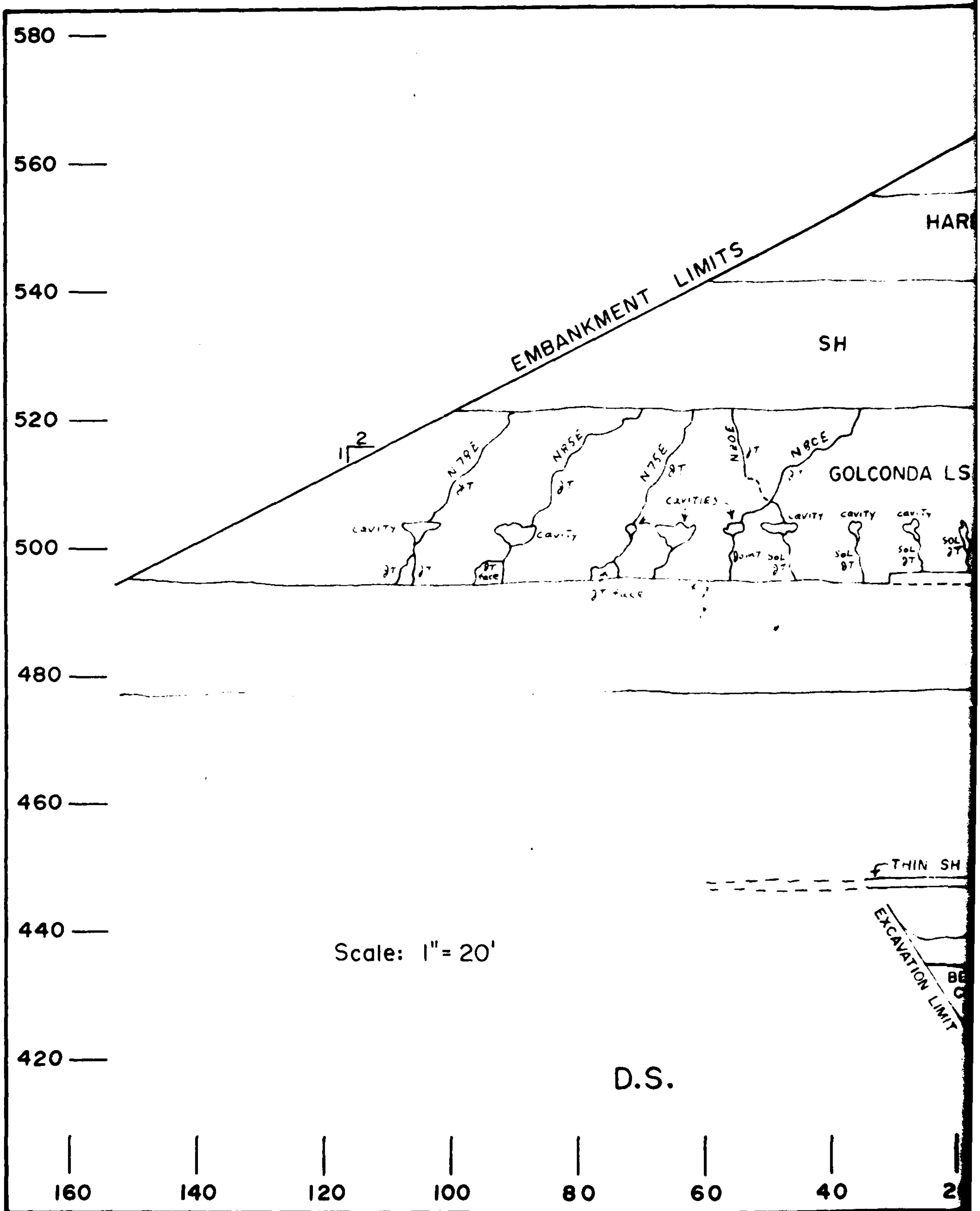
℄
TOP OF DAM
SH
SH
GLEN DEAN LS
JT. face N5°W
JT. N 75°W
JT. face N85°W
JT.
highly (w) JT. face N31°W
JT. face N 79°W
JT face N16°W
clay filled cavity
JT face
JT face
clay filled Sol B/P
clay filled Sol B/P
clay filled Sol B/P
EMBANKMENT LIMITS
2
1
HARDINSBURG SH
JT. N85°E
JT. N 90°W
GOLCONDA LS
N 70°W
Sol cavity along JT
cavity
Sol JT. face N 80°W
Sol JT face N 15°W
Shaley (w) LS
JT face N30°W
JT face N 20°W
SH seam
SH seam
fallout
cavities
cavity
JT face
JT face
Sol Cavity
sol cavities
cavities
Presplit corner
GOLCONDA SH
BIG CLIFTY SS
SOFT SAND SEAMS
CUTOFF EXCAVATION LIMITS
CUTOFF EXCAVATION LIMITS
BEECH CREEK LS
cavity
cavity
B/P
JT face N 70°W
JT face N 20°W
JT face
JT face
JT face
B/P
SH
BASE OF EXCAVATION
D.S.
100
120
40
20
℄
20
40
60
80
PAT
DAM
LT. ABU



PATOKA LAKE
DAM FOUNDATION
LT. ABUTMENT WALL MAP

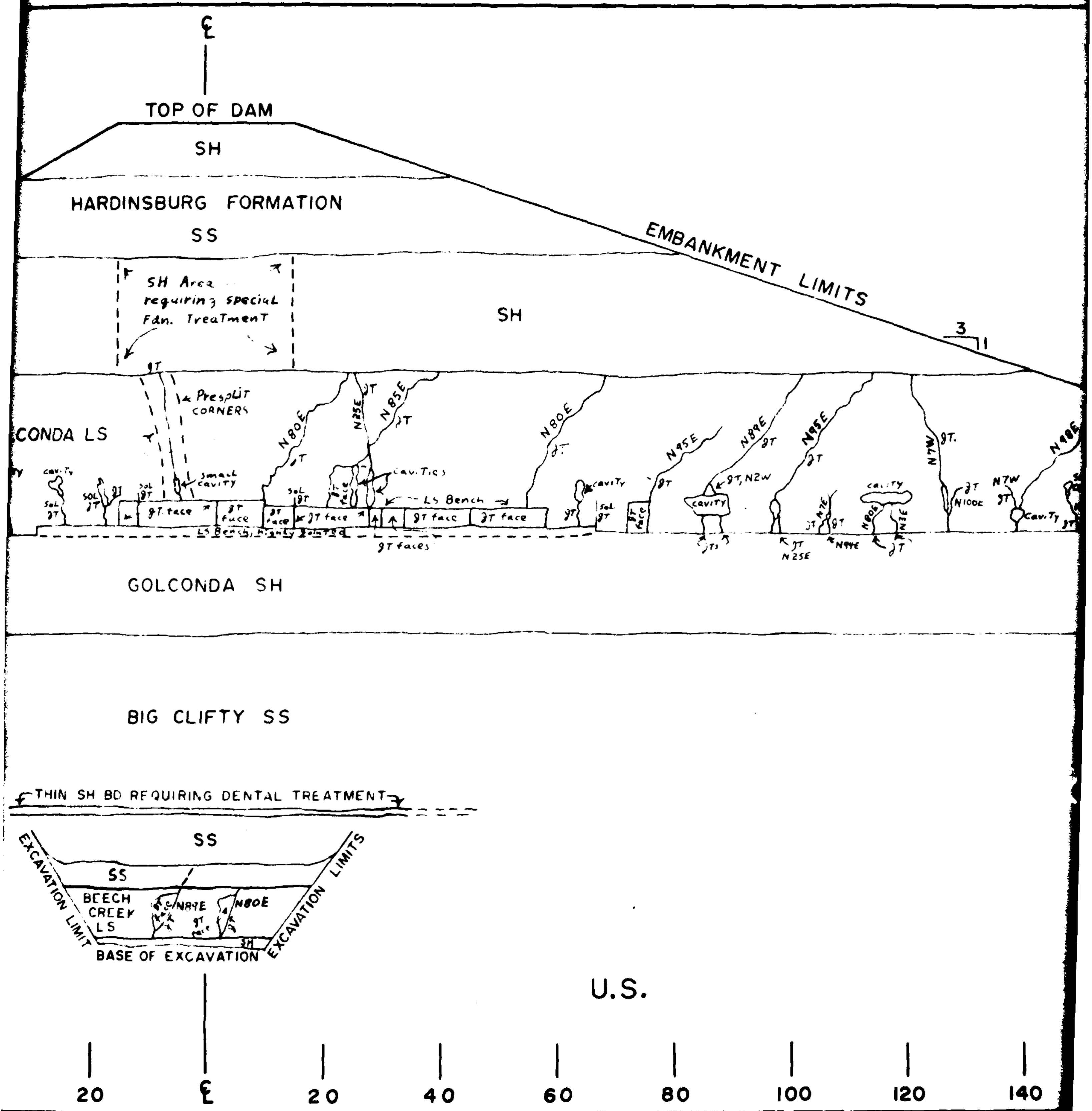
FIGURE IX-1

2 1 3



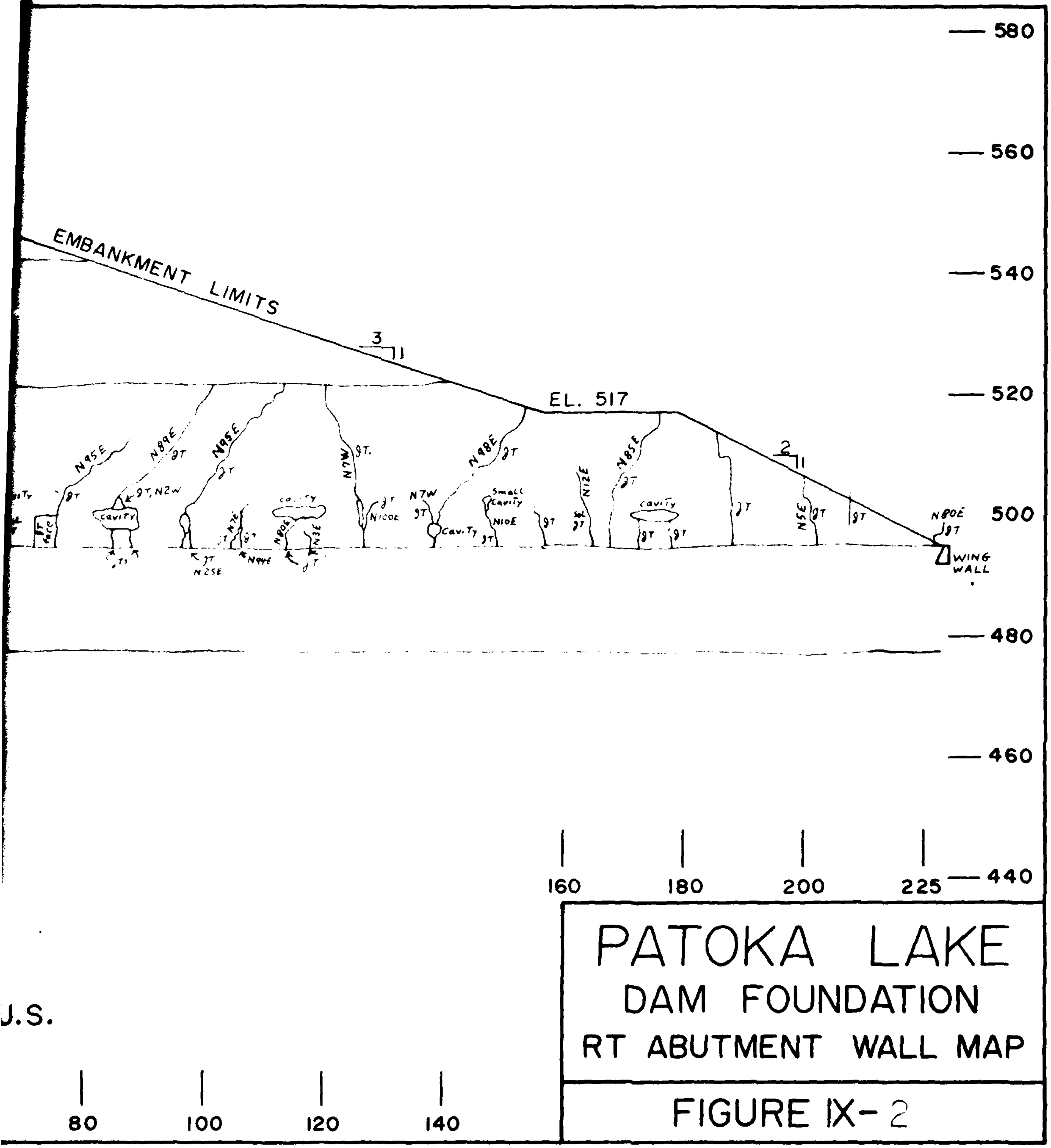
580
560
540
520
500
480
460
440
420
EMBANKMENT LIMITS
1
2
HAR
SH
GOLCONDA LS
CAVITY
CAVITIES
THIN SH
EXCAVATION LIMIT
Scale: 1"= 20'
D.S.
160
140
120
100
80
60
40

1



TOP OF DAM
SH
HARDINSBURG FORMATION
SS
EMBANKMENT LIMITS
SH Area requiring special Fdn. Treatment
SH
3
1
Presplit CORNERS
CONDA LS
N80E
N25E
N85E
N95E
N89E
N7W
N98E
cavity
small cavity
Cavities
LS Bench
jT face
LS Bench, Highly Jointed
jT faces
N2W
N100E
N25E
N94E
GOLCONDA SH
BIG CLIFTY SS
THIN SH BD REQUIRING DENTAL TREATMENT
SS
SS
BEECH CREEK LS
N80E
SH
EXCAVATION LIMIT
EXCAVATION LIMITS
BASE OF EXCAVATION
U.S.
20
20
40
60
80
100
120
140

2



PATOKA LAKE
DAM FOUNDATION
RT ABUTMENT WALL MAP

FIGURE IX- 2

B. Spillway

9-15. Basically, three types of materials were found in the spillway foundation: overburden, sandstone and limestone. The overburden, besides being found at the top of the side slopes, was found at the outlet portion of the spillway. The overburden in this area consisted primarily of rock colluvium. Some quantities of clay were found within the rock fragment matrix. This overburden type was derived mostsly from weathered sandstone and therefore mirrored some of the parent rock characteristics. This type of overburden apparently graded into the parent rock. The spillway floor from station 16+25 to the end on the left side, and from station 21+75 to the end on the right side, was formed by overburden material. The material was generally hard, dry, and able to maintain the finished grade and slopes following excavation. The right slope in the overburden type material was changed to a straight 3 to 1 slope when no rock was encountered. Some contamination of the overburden material by rock fragments occurred during final dressing, but did not alter the characteristics of that material. All slopes and grades in overburden were seeded upon completion. Little washing has occurred since construction except in the area of the paved ditch that crosses the spillway floor.

9-16. The majority of the spillway side slopes were excavated in the Mansfield Sandstone. This rock consisted primarily of sandstone with numerous soft shale interbeds. Varying degrees of hardness were encountered according to rock types and degree of cementation. Excavation was generally not difficult and most of the excavation could have been performed without explosives as occurred at the inlet end of the spillway. All spillway side slopes in rock were presplit. However, the side slopes in the cutoff trench area were not presplit. From the stable, clean condition of these cutoff trench side slopes, presplitting of the spillway side slopes was not deemed necessary. The resultant spillway presplit side slopes remained in good stable condition with only minor spalling during freeze and thaw conditions. In most cases, presplit hole casts could still be clearly discerned long after construction was complete, further indicating stable side slopes. Little groundwater was encouontered in the sandstone. Some groundwater was encountered during blast hole drilling, but was not evident in the sandstone following excavation.

9-17. As excavation in the sandstone progressed, numerous hollow collapse structures were encountered. As discussed earlier in this report, these structures were formed by the collapse of overlying sandstone into the solutioned limestone below. Most of these structures took the form of large dome-shaped cavities with some disturbances on the side walls. Several of these structures have been preserved on the spillway side walls between stations 1+0 and 4+0. In these instances where the structures were formed on the side walls, the normal rock bedding structure has been disrupted and subsequent side slopes have been weakened and rendered less stable. In most cases, however, the collapse structures were noted inside the excavation and side slope limits. In those instances, the collapse zones were filled with rubble during excavation for safety reasons. As a result, none of the collapse structure characteristics were noted on the final spillway floor.

9-18. The softness of the sandstone caused many problems during embankment construction. By the time the sandstone was excavated and transported to the embankament, it would deteriorate to fine sand with small rock fragaments. The sandstone portion of the final graded spillway floor also encountered problems due to the softness of the rock. Much shallow overbreakage with resultant overexcavation by equipment occurred in the sandstone portion of the spillway floor. When this occurred, the low areas were filled in and final graded with motor graders. This overbreakage and filled collapse areas tended to obliterate the bedding characteristics of the floor rock. Consequently, much of the spillway floor was graded to a sand composition which would erode badly if the spillway was ever used. Apparently some connection between the Hall Spring and the spillway floor existed as noted by diesel fuel traces in the spring during periods of heavy rain. However, no erosion or subsidence has occurred in the spillway floor due to this pattern of water flowage.

9-19. The third type of foundation material in the spillway excavation was the upper portion of the Glen Dean Limestone. This rock outcropped on the right side of the spillway between stations 14+50 and 21+75. In this area, the thin 3-foot shale overlying the limestone was intact with a minimum of weathering. As discovered in other portions of the project, this thin shale protected the underlying limestone from vertical movement of groundwater and the resultant extensive solutioning along vertical joints. Nonetheless, the Glen Dean Limestone occurred near the old rock-overburden boundary. Consequently, some erosion of the limestone occurred and a rudimentary "pinnacle" type structure was beginning to form near the outer contacts. The majority of the limestone, however, remained in good condition with staining and weathering limited to vertical joints.

9-20. The hardness of the limestone and the rudimentary "pinnacle" structure made the construction of the final spillway floor grade extremely difficult. Tolerances on the final floor grade were very close and required much extra drilling and blasting to excavate this grade. All presplit holes and most blast holes were subdrilled approximately 6 inches below grade. The presplit lines held true, but the bottom surface in the limestone portion of the spillway floor was very irregular. Consequently, the contractor was required to reshoot the humps to a shallow depth. This process was repeated many times until the final grade tolerances were met. As in the case of the sandstone portion of the spillway floor, all low areas and natural depressions in the limestone were filled with fragmented sandstone and graded to final tolerances. The resultant surface was smooth, but the rock surface was not.

9-21. Along the berm upstream of the concete sill, a portion of the Glen Dean Limestone was encountered also. Portions of the limestone were also exposed from approximate stations 1+0 to 4+0. In this area, a thicker (up to 6 ft.) sections of the overlying shale-limestone member of the Glen Dean Formation was exposed. The 50-foot wide bench was founded mostly in this upper section. Little excavation of the limestone was performed. The same methods and results were used to excavate this portion of the spillway limestone as were used in the limestone downstream of the concrete sill.

9-22. The remaining portion of the 50-foot berm created by the spillway widening was founded mostly in sandstone. No troubles were experienced during construction of this berm to grade. The condition of the rock in the area of the concrete sill has been discussed in the cutoff trench section of this report, since the sill was formed on top of the completed trench.

C. Dike

9-23. Most of the foundation materials ranging from overburden to limestone were also encountered in the dike foundation. The condition of the various foundation materials varied with their relative locations within the dike foundation.

9-24. The characteristics of the overburden varied widely depending upon their positions within the foundation. None of the dike was founded on overburden, but the deep overburden side slopes caused concern during construction. All of the overburden formations were encountered in the dike area, mostly in the valley bottom. The Prospect and colluvium soils caused no problems and the 1.5 horizontal to 1 vertical sideslopes remained stable in these formations. Difficulties in the silts, sands and clays of the Atherton Formation caused much concern in the deepest part of the valley bottom. Originally these lacustrine soils did not require excavation. However, during the deep excavation of the left abutment, these soils were encountered. The resulting unstable foundation conditions and excessive groundwater necessitated their removal. The side slopes in this zone were changed to 1 horizontal to 1 vertical due to the constraints on the higher downstream side slopes. Pits were excavated and slotted casings with submersible pumps were installed to dewater and stabilize this material. The final excavation was completed through the lacustrine deposits to the underlying shale with a dragline. The resulting sideslopes were marginally stable and the embankment in this zone was completed as quickly as possible. The lacustrine deposits were characterized by alternate layers of loose wet sand, silts, and minor amounts of clay. A thin stable cemented gravel zone was encountered at the very base of the excavation on the left side.

9-25. The highest rock encountered in the dike foundation excavation was the sandstone member of the Mansfield Formation. The characteristics of this rock were entirely different on the left and right abutments of the dike. On the left abutment this rock was fairly well cemented and quite competent. The 1V to 1H side slopes excavated in this rock of the left abutment remained stable with a minimum of spalling through one extreme winter of construction.

9-26. The sandstone encountered on the right abutment of the dike possessed entirely different characteristics than the same rock on the left abutment. The right abutment sandstone possessed a very fine sugary texture with extremely poor cementation, probably as a result of leaching. The 1V to 1H side slopes of the original excvation were so unstable that they were changed to 1 vertical to 1.5 horizontal to maintain stability. Cementation and the physical competence of the sandstone improved further upstation and deeper into the right abutment.

9-27. Lower in the sandstone near the limestone contact, cementation and slope stability improved, but other problems occurred. At the extreme right end of the dike excavation, a collapse structure similar to those occurring in the spillway was encountered. The sandstone in this area was highly fractured, blocky and unstable. Extensive treatment and design changes were required to stabilize this area.

Additionally, an alternate right abutment tie-in of reinforced concrete was constructed to overcome the poor condition of the sandstone at this location. The lower side walls in sandstone remained stable during dike and cutoff trench construction.

9-28. The next lower rock type encountered in the dike foundation was the Glen Dean Limestone. Again, the physical characteristics of this rock varied according to its position within the dike foundation. Generally the rock became more competent further back into the abutments. The limestone encountered on the right abutment of the dike was highly solutioned and eroded during the interval of exposure between Mississippian or Pennsylvanian intervals of deposition. A resultant "pinnacle" type structure was formed in most of the limestone on the right abutment, stations 8+0 to 17+0. This structure, consisting of large isolated limestone blocks surrounded by Terra Rosa clays, caused many difficulties during construction. Further complicating this pattern was the erosional removal of the limestone in a stair-stepped pattern along bedding planes. Modifications in design and treatment, in addition to removing a portion of the limestone down to shale, were required to overcome this extensive erosional pattern in the limestone. The final vertical side slopes were highly weathered, solutioned and irregular. One large cavity causing an overhang was present on the upstream wall between stations 17+15 and 17+60. In other areas, large blocks of limestone were removed along joints and bedding planes changing the original straight side walls into irregular geometric patterns. Extensive patterns of clay-filled solutioned joints remained in the side walls. Occasional cavities and overhangs required further treatment. The remaining rock was weathered and stained to varying degrees throughout. When the excavated limestone could be separated from the surrounding clay, it was further broken and used in the upstream dam slope protection.

9-29. Limestone conditions further into the right abutment of the dike improved. The "pinnacle" effect gradually decreased, but extensive solutioning along joints and bedding planes continued. When the final excavation end point was reached at station 18+0, the condition of the limestone was unsuitable for forming a tie-in for the embankment. The limestone was disected primarily by two vertical, wide, clay-filled solutioned joints running at low angles to the dike centerline on the upstream and downstream corners of the excavation. In addition, smaller patterns of vertical and horizontal clay-filled solutioned cavities further complicated the stability of the tie-in section. Some groundwater flowed from these patterns of solutioned joints and bedding planes. This extensive pattern of solutioning led to the collapse structure noted in the overlying sandstone. To overcome these problems, a reinforced 4-foot thick concrete wall was constructed over the solutioned limestone at the tie-in point. The top of the limestone was highly irregular and the thin protective shale was removed for the most part.

9-30. The limestone from stations 3+15 to 1+0 on the left abutment displayed a similar "pinnacle" structure to that found on the right abutment of the dike, but the left abutment structure was less highly developed than on the right abutment. Two distinct stairsteps occurred at stations 2+70 and 1+0. The limestone between stations 3+15 and 1+0 was weathered and solutioned to the base of the limestone with numerous clay-filled solutioned joints at varying angles. The limestone between stations 1+0 and 0+40 was intact and reached its full thickness. The upper founding surface was potholed and slightly irregular. Two vertical joints ran at low angles to the centerline in this full section of limestone. The thin protective shale was present between stations 1+0 and 4+0, but was only 0.5 foot thick and weathered to clay. Apparently, this short section of complete limestone was buried deep enough to protect it from severity of erosional forces that dissected the limestone beyond station 1+0.

9-31. The deepest rock occurring in the dike foundation was the Hardensburg Shale. This rock was nearly uniform in physical properties from one end of the dike to the other end. This rock unit was impervious and formed the majority of the foundation for the dike embankment.

The rock was slightly irregular in texture with numerous compaction slickensides. One micro monoclinic structure was noted at station 3+25. Bedding was not well defined which was characteristic of this type of compaction shale. The exposed surfaces in the deeply eroded valley bottom were not affected by weathering. This rock dried and deteriorated rapidly and required wetting before embankment placement. Colors varied from greenish-grey to mottled red and greenish-grey.

D. Dike to Spillway

9-32. The experimental grouting program between the dike and the spillway resulted in a good understanding of the character of the rock in this area prior to the construction of the cutoff wall. Basically, conditions in this area were similar to the rock conditions encountered at the right abutment of the dike. Large voids encountered between the Mansfield Sandstone and the Glen Dean Limestone probably resulted from collapse structures similar to those observed on the dike right abutment and in the spillway excavation. The sandstone, where intact, was moderately cemented and competent as judged from dike, cutoff trench, and spillway exposures. During cutoff trench construction, a total of four large collapse structures were encountered on the downstream wall between the dike and the spillway. (Reference Plates IX-1 & IX-2.)

9-33. The limestone, however, was highly dissected by vertical and horizontal solutioning similar to that found on the right abutment of the dike. The condition of the limestone improved toward the spillway as the thickness of the thin protective covering shale increased in that direction. Many of the grout holes were abandoned in this area, furthering this conclusion. As observed during cutoff trench construction, the solutioning of the limestone did decrease significantly until reaching the spillway end of the trench. No large cavities were discovered in the limestone, but many small cavities were noted in the cutoff trench walls. A WES program to determine the orientation of cavities and solutioned joints resulted in little clarification of open joint patterns. This report has been listed in the bibliographic appendix to the foundation report as further reference material.

E. Spillway to Dam

9-34. The condition of both the Mansfield Sandstone and the Glen Dean Limestone apparently improved considerably between the spillway and the left abutment of the dam. No direct observations of either formations were made in this area, but the evidence from the exploratory grouting program lead to the above conclusion. The sandstone observed on the left abutment of the dam and the right side slope of the spillway appeared to be moderately cemented with little or no leaching as found in other areas of the foundation. No sandstone collapse zones were encountered during the exploratory grouting program in this area, although two collapse zones were noted on the right spillway cut upstream at the control site.

9-35. The protective shale in this area averaged 2 to 3 feet thick. This increased thickness was apparently enough to protect the Glen Dean Limestone from extensive solutioning. Of the large number of exploratory grout holes drilled in the limestone, few required grouting and only one hole was abandoned after pumping 500 cubic feet of grout. This data further substantiated the relatively sound condition of the Glen Dean Limestone in this area. Occasional open vertical solutioned joints were present as noted on the left abutment of the dam, but the limestone as a whole appeared to be sound in this area. As a result of the low grout take in this area, the decision to postpone further foundation treatment in this area was made.

F. Water

9-36. The occurrences of freewater varied with the types of rocks. Generally, freewater flowed near the base of the limestones at the lower shale contacts. Little or no free water was encountered in the sandstones. The shales were impervious.

9-37. The largest quantities of free-flowing water were encountered in the Beech Creek Limestone portion of the dam excavation, especially during first exposure. Both abutment outcrops produced water and required extensive water control methods during construction. Sump pumps were installed in low areas upstream and downstream of the core trench to remove the excessive water flowing from this rock. In particular, one large spring was noted on the left abutment at the upstream toe of the exposed limestone. Most of the water flowed from vertical open and solutioned joints in the limestone and was stopped when these joints were sealed with concrete. However, free water flowing from one of the large upstream joints could not be stopped in this manner and pipes were installed through the dental concrete to allow drainage. In the area of this spring, filter gravel was installed with a sump pump and the excess water was pumped until the embankment could be completed above this zone of seepage. Additional slotted casings with submersible pumps were installed at several points in the gravel filters within the core trench area and in other areas of the foundation. These pumps were operated until the embankment could be completed above these gravel filter zones. The pumps were then removed, the casings were filled with gravel, and sealed with concrete as described in the dam foundation treatment section of this report. Proportionately more freewater was observed upstream of the cutoff trench than downstream.

9-38. Slight amounts of seepage were noted in the Big Clifty Sandstone. This seepage was due to the porosity of the rock. Slight seepage was noted along some bedding planes. This seepage stopped as construction progressed. Seepage from this rock did not produce an appreciable amount of water and did not require any seepage control during construction.

9-39. The Golconda Limestone contained varying amounts of free flowing water during construction. Due to the dip, the exposed Golconda on the right abutment of the dam produced proportionately more freewater than was produced in this formation on the left abutment. The flow pattern on the right abutment took the form of small springs exiting at the lower limestone contact through open joints along the entire exposed face. These springs, for the most part, flowed year around. Some migration of these springs from upstream to downstream of the dam centerline was noted during grouting operations. Following the concrete wall construction, this freewater flow pattern was again altered by circuit grouting behind the wall. All flowage through the wall pipes was stopped. Consequently, water was forced around the downstream end of the wall into the excavation. During final stages of dam construction, an underdrain was constructed (Mod P00044) to discharge this flowage into the stilling basin. A weir was installed in this underdrain and was monitored during impoundment of the lake. Data from this monitoring indicates that the flow from this drain varies directly with the amount of rainfall and that the fluctuations of the lake level have had no effect upon the flow from this drain. (Reference Figure IX-3.)

9-40. Free-flowing groundwater in the Golconda Limestone on the left abutment of the dam diminished rapidly after drainage of the newly exposed clay-filled cavities in this rock. After cleaning and just prior to construction of the concrete wall at this location, freewater was limited to very small seeps occurring at the base of the limestone in vertical solutioned joints. No additional control measures were necessary. No other exposures of this limestone were encountered in the damsite foundations.

9-41. Free-flowing water from the Glen Dean Limestone on the left abutment of the dam was negligible after final excavation. Only small

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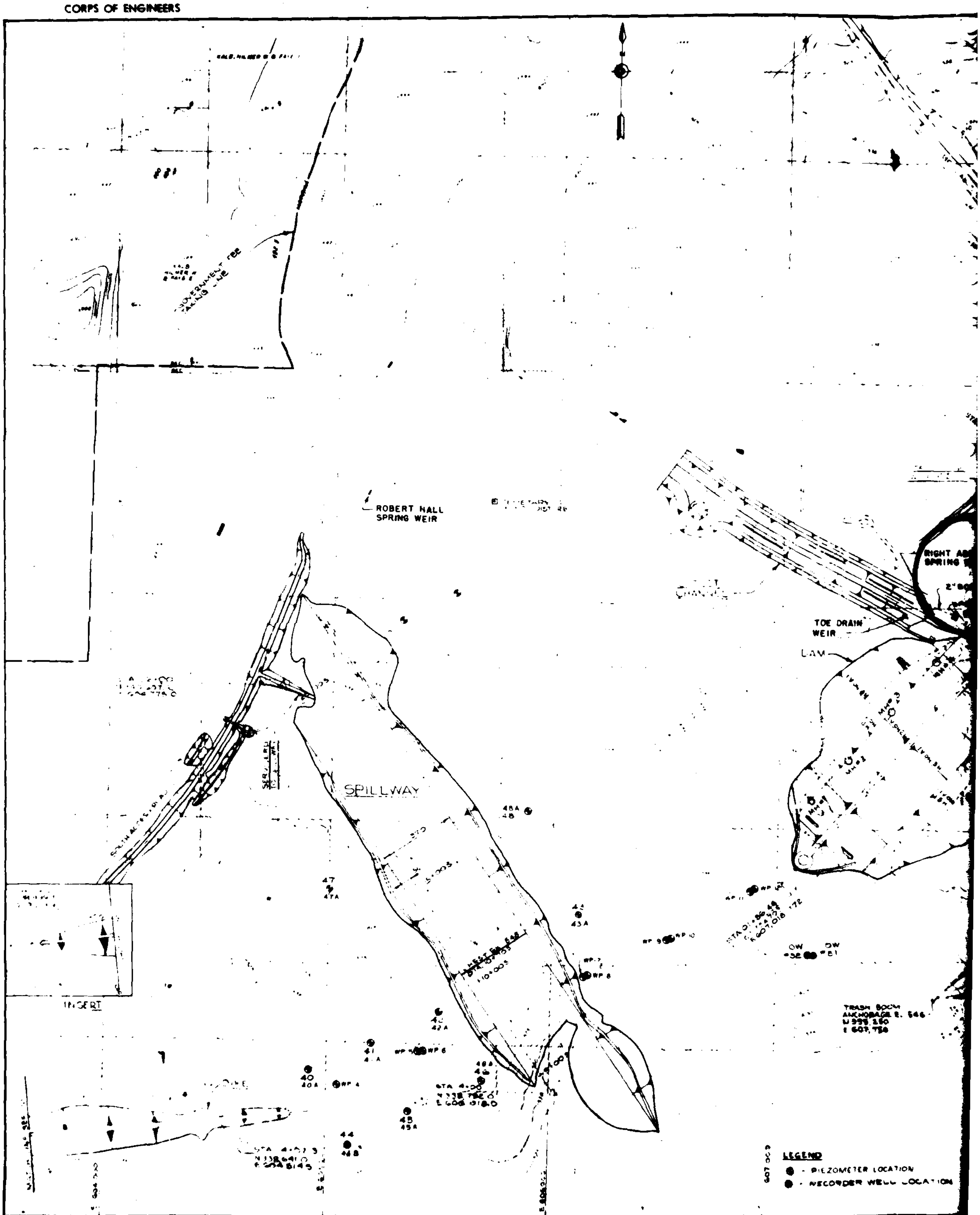
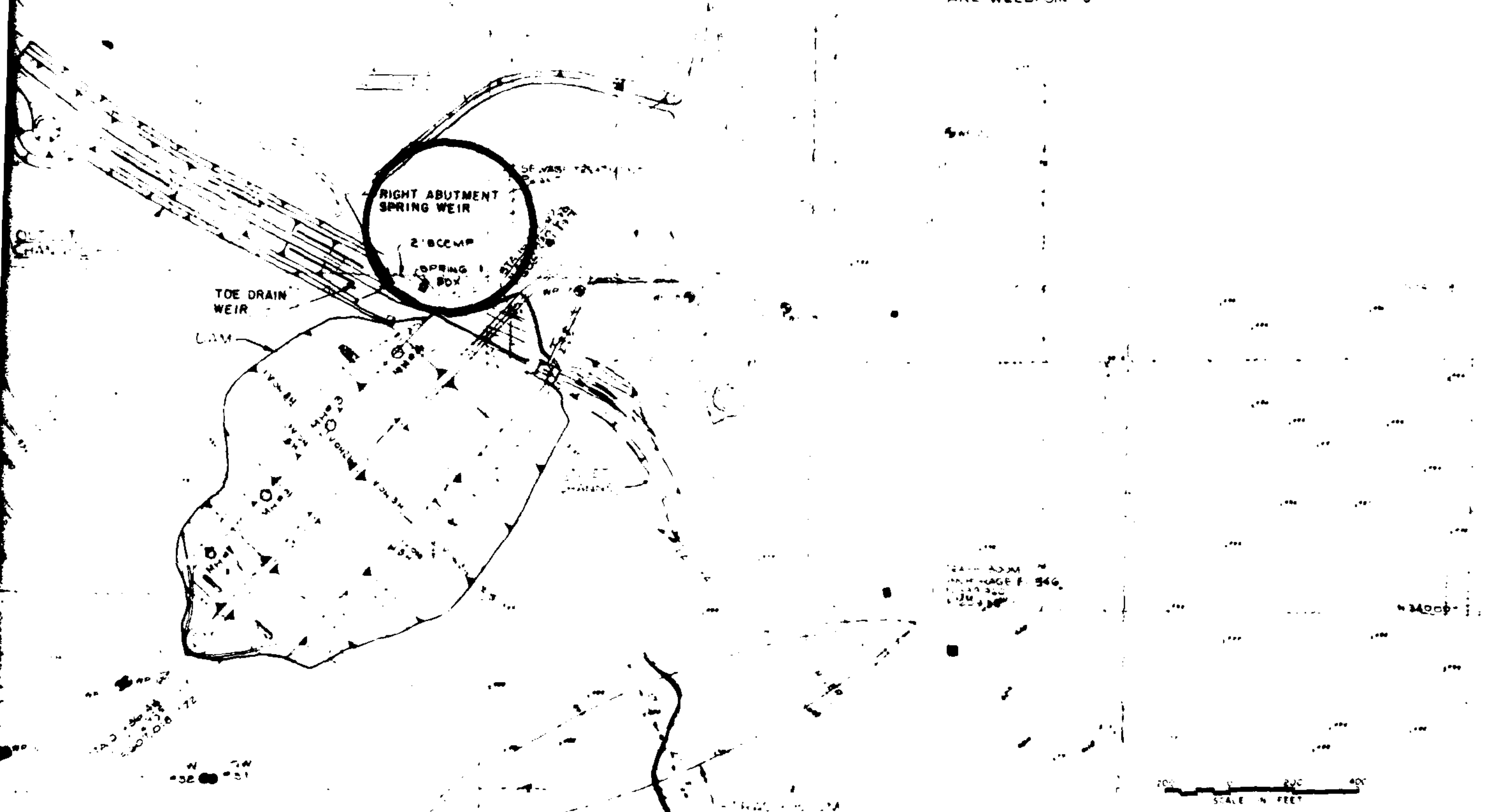


U.S. ARMY

PIEZOMETER SCHEDULE

NO.	STATION	OFFSET	TIP EL	TOP EL	FORM. OBS.
40	10+80 SPILLWAY	875 LT	460.6	[illegible]	LS-GOL
40A	10+80 SPILLWAY	868 LT	507.9	[illegible]	LS-GD
41	10+30 SPILLWAY	860 LT	457.8	562.3	LS-GOL
41A	10+30 SPILLWAY	870 LT	508.8	564.4	LS-GD
42	9+80 SPILLWAY	220 LT	464.7	551.6	LS-GOL
42A	9+80 SPILLWAY	230 LT	515.9	551.4	LS-GD
43	10+00 SPILLWAY	540 RT	470.3	630.1	LS-GOL
43A	10+00 SPILLWAY	550 RT	522.4	629.8	LS-GD
44	7+00 SPILLWAY	900 LT	464.6	551.6	LS-GOL
44B	6+30 SPILLWAY	900 LT	[illegible]	552.1	LS-GD
45	6+80 SPILLWAY	600 LT	459.5	580.0	LS-GOL
45A	6+80 SPILLWAY	610 LT	510.9	580.3	LS-GD
46	6+60 SPILLWAY	250 LT	463.7	606.2	LS-GOL
46A	6+90 SPILLWAY	250 LT	517.4	608.0	LS-GD
47	7+20 SPILLWAY	330 LT	444.0	571.9	LS-GOL
47A	7+30 SPILLWAY	330 LT	502.3	572.5	LS-GD
48	15+30 SPILLWAY	605 RT	468.2	641.5	LS-GOL
48A	15+40 SPILLWAY	605 RT	521.2	642.0	LS-GD
49	28+30 SPILLWAY	610 RT	513.9	616.4	LS-GD
50	25+00 SPILLWAY	880 RT	516.5	625.7	LS-GD
SW-5	40+20 GROUTLINE	330 RT	525.5	649.45	LS-GD
SW-52	40+10 GROUTLINE	310 RT	473.5	648.03	LS-GOL
WP-4	20+10 GROUTLINE	40 DS	508.8	574.4	LS-GD
WP-5	26+20 GROUTLINE	04 DS	509.9	608.9	LS-GD
WP-6	26+38 GROUTLINE	74 DS	469.2	610.2	LS-GOL
WP-7	32+30 GROUTLINE	80 DS	470.2	604.9	LS-GOL
WP-8	32+40 GROUTLINE	50 DS	522.8	606.9	LS-GD
WP-9	36+40 GROUTLINE	50 DS	524.5	632.7	LS-GD
WP-10	36+50 GROUTLINE	50 DS	47[illegible].7	632.5	LS-GOL
WP-11	40+40 GROUTLINE	50 DS	526.[illegible]	643.6	LS-GD
WP-12	40+50 GROUTLINE	50 DS	478.4	642.8	LS-GOL
WP-17	59+30 GROUTLINE	30 DS	490.6	570.9	LS-GOL
WP-18	62+70 GROUTLINE	30 DS	497.1	567.1	LS-GOL
WP-19	65+70 GROUTLINE	20 DS	495.0	569.3	LS-GOL
WP-20	75+00 GROUTLINE	50 DS	501.8	574.6	LS-GOL

NOTE
PIEZOMETERS 40-50 ARE CASAGRANDE. 5 & 52 ARE RECORDER WELLS. WP-4 - WP-12 & WP-17 - WP-20 ARE WELLPOINTS.



LEGEND
● - PIEZOMETER LOCATION
● - RECORDER WELL LOCATION

REVISION	DATE	DESCRIPTION	BY

U.S. ARMY ENGINEER DISTRICT LOUISVILLE
CORPS OF ENGINEERS
LOUISVILLE KENTUCKY

DESIGNED
DRAWN TRACED
CHECKED
SUBMITTED

WABASH RIVER BASIN
PATOKA LAKE
PATOKA RIVER, IND
DAM & SPILLWAY
ABUTMENT PIEZOMETERS

APPROVED
SCALE 1" = 200'

PLATE

FIGURE IX-3

2

seeps were noted along vertical joints at the base of the limestone within the excavation. No control of this freewater was necessary. This lack of seepage was related again to the dip of the limestone which generally dipped away from the excavation at approximately 5 degrees. This limestone was not exposed on the right abutment of the dam.

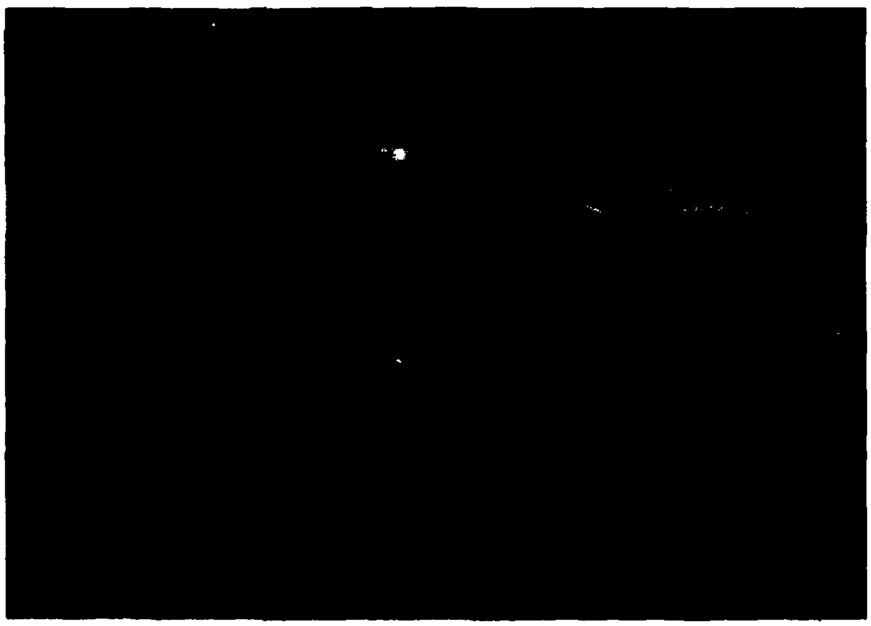
9-42. During excavation and construction of the cutoff wall between the dike and the spillway, excessive amounts of free groundwater were encountered in the lower portion of the Glen Dean Limestone. Portions of the cutoff trench had to be pumped continuously to allow construction to progress. Wet mud within the limestone caused problems during drilling and blasting operations. Again, the majority of the freewater was encountered near the base of the limestone. When left unpumped over weekends, open portions of the trench would fill to varying depths up to two-thirds the depth of the trench. The majority of the trench was excavated during the winter months when groundwater levels were known to rise. Conversely, the test portion of the cutoff trench was excavated during the dry season and less freewater was excavated. The dip of the limestone formation facilitated the flow of groundwater into this area. Further discussion of this water occurrence can be found in Section VIII of this report.

9-43. Excavations in the Glen Dean Limestone portion of the dike foundation encountered varying amounts of freewater. As noted in the special grouting report (Appendix E) the original groundwater level at elevation 524 was recorded during grouting and exploratory drilling of the left abutment. Following excavation of the limestone at the left abutment of the dike, little freewater was encountered. Occasional small seeps were noted along vertical joints at the base of the limestone. A small sump pump at station 2+60 was installed in the downstream gravel filter and easily controlled this seep water. At the right abutment of the dike, proportionately more freewater was encountered in the limestone during excavation. Occasional small seeps were encountered along joints and bedding planes in the lower portion of the limestone. At the extreme right end of the excavation, considerable freewater was encountered low in the limestone when the system of extensive solutioned cavities was uncovered. This water was controlled by sump pumps located at the extreme end of the excavation. Again, more freewater was encountered during initial excavation in the winter months than during final construction in the summer months.

9-44. As expected, very little freewater was encountered in the Mansfield Sandstone overlying the various dam structures. A very slight amount of seep water was encountered at the sandstone-limestone contact along the backwall of the dam left abutment (station 0+35). During blasting operations in the spillway, some freewater in the sandstone was encountered in the blast holes requiring the use of a water gel explosive. Upon completion, the sandstone within the spillway remained dry except for a slight amount of water continuing to seep at the sandstone-limestone contact near the outlet end. All other portions of the foundation in sandstone remained dry, including the two abutments of the dike.

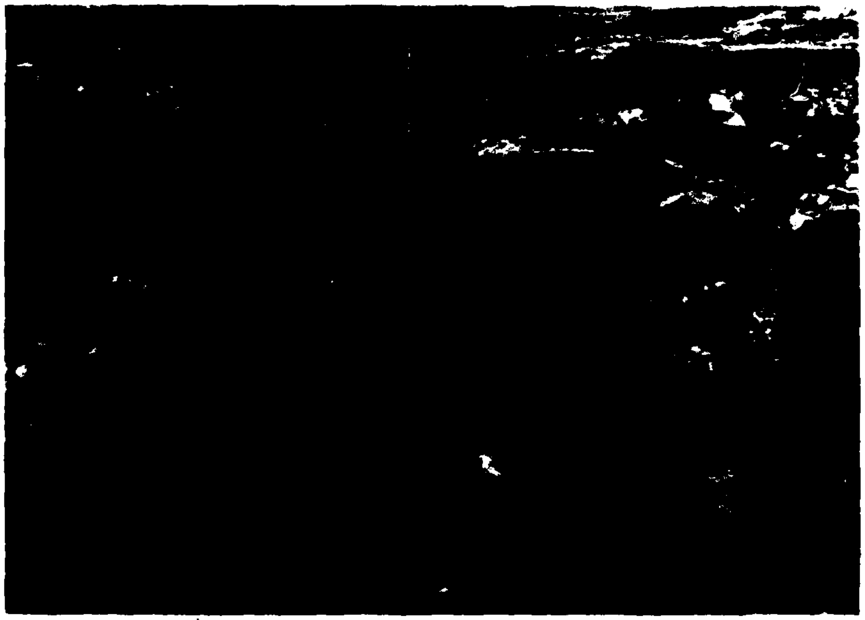
9-45. Freewater occurrence and dewatering of the thick saturated overburden within the dam foundation has been described in the dam excavation section of this report. Small amounts of seepage were noted at the overburden-rock contact at the base of the dam excavation. However, this seepage was reduced as the overburden drained and did not require additional seepage control. During excavation of the dike foundation through the overburden portion of the old river channel, excess freewater was encountered in the deep buried lacustrine sand and silt deposits. This water was controlled by sump pumps and diminished as the side slopes drained during construction progress. Original groundwater levels in the overburden at both the dam and the dike corresponded closely with the original ground elevations.

2 September 1976
Dam Left Abutment, Golconda Limestone. Cleanup in preparation for concrete wall. Note: removed clay cavity filling material at lower right corner.



31 March 1976
Dam Cor Trench. Solutioned joint pattern in Beech Creek Limestone prior to core trench excavation.

Plate IX - 1.



15 April 1976
Beech Creek Limestone. Outcrop downstream of dam core trench. Note: contact, black weathering of limestone and dark grey cavity filling.

2 October 1976
Dam Left Abutment, Upstream End of Golconda Limestone Wingwall. 1 ft. concrete wall will end at stake in upper left corner of photo.

Plate IX - 2.

X DAM FOUNDATION TREATMENT

10-01. As the dam excavation phase of construction neared completion, foundation problems and the need for extensive treatment was realized. Foundation treatment conformed to the different rock types and discussion will therefore be grouped according to rock types in ascending order.

10-02. The lowest rock member in the dam foundation was the Elwren Shale. Grouting of the single row of holes on 20-foot centers has been discussed in the grouting section of this report. As indicated during grouting operations, an additional 2 feet of soft shale, which was disturbed during excavation, was removed in the left abutment portion of the cutoff trench. No additional treatment other than normal clean-up was deemed necessary. This same formation formed the foundation for the portion of the dam in the cutoff trench. The shale surface was cleaned by brushing and air jetting. This rock was wetted before placing the impervious portion of the embankment.

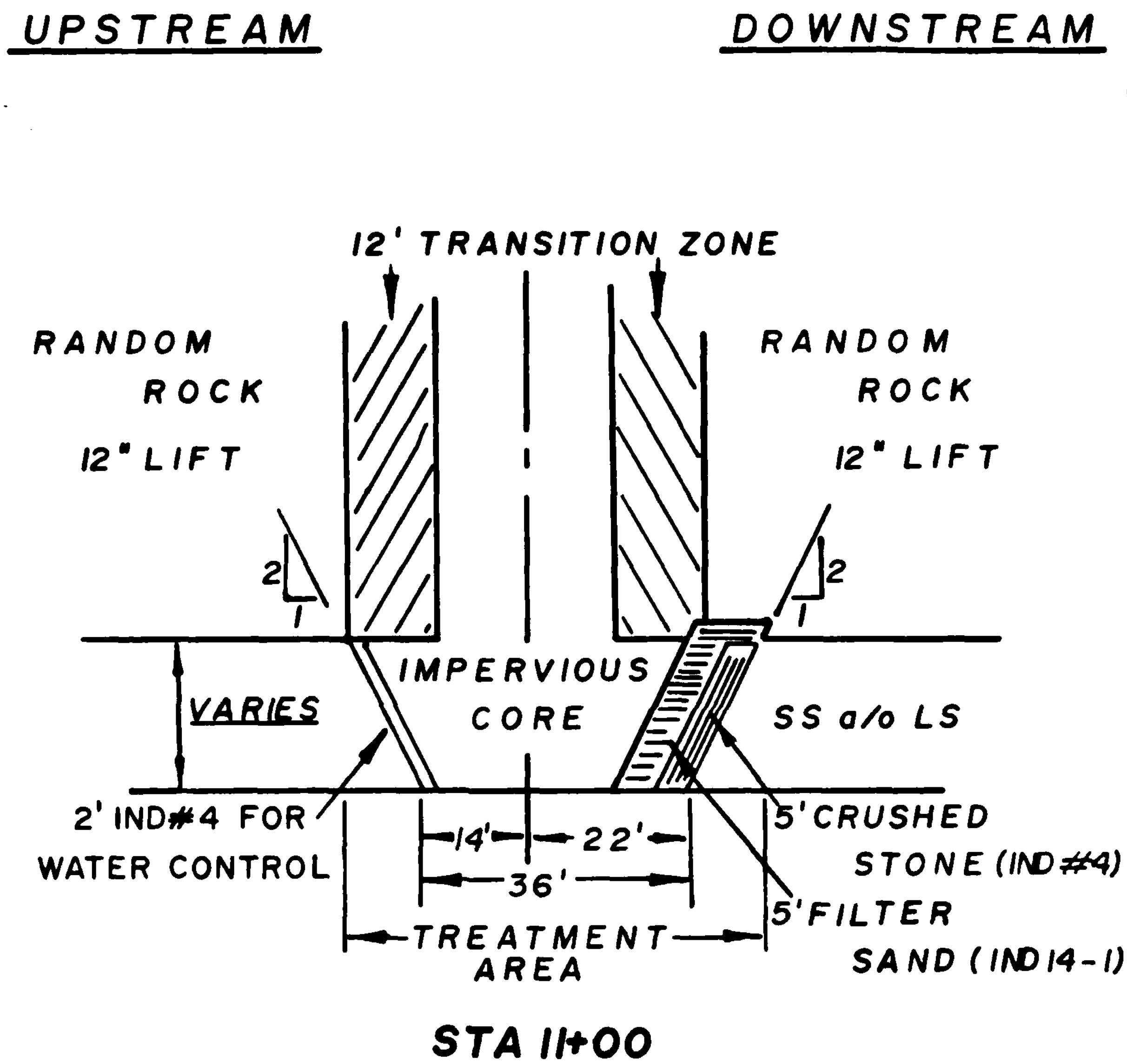
10-03. The next rock in ascending order, Beech Creek Limestone, required extensive treatment which in turn required modification of the original contract. This rock was found to be extensively solutioned throughout the foundation area, especially at the contact with the overlying sandstone. Once this solutioned contact zone was exposed during excavation, large quantities of water were produced which basically dewatered both abutments within the limits of the excavation. The freewater accumulated in low portions of the foundation and was pumped outside the excavation limits from there. Since this solutioned zone was continuous along both abutments, a method of protection against piping and breeching of the embankment materials was mandatory. After foundation cleanup, all cavities and caves in the Beech Creek Limestone were cleaned to a depth three times the width and filled with concrete upstream and downstream of the core trench. A 10-foot wide double zoned reversed filter of natural sand and crushed limestone aggregate was placed between the limestone and the embankment materials for additional protection against piping. The reverse filter was continuous along both abutments upstream and downstream of the core trench, in the solutioned limestone area. This reverse filter was similar to the one constructed along the downstream wall of the dam cutoff trench as shown on Plate X-3.

10-04. The original contract required a positive cutoff excavated through the Beech Creek Limestone into the underlying shale. The severity of the solutioning found in the Beech Creek Limestone necessitated extending this core trench significantly in both directions in order to expose sound rock. At the same time the trench was widened from 24 to 36 feet. To protect the impervious core in the cutoff trench, concrete walls were constructed over the limestone at both ends of the trench. These walls were reinforced with (6 x 6 inch) #4 gage wire mesh. Grouting preceded the wall installation at station 7+0 and the solutioned cavity at this location was effectively sealed. Grouting also preceded construction of the concrete wall at station 12+50, but extensive solutioning was completely removed before grouting at this end of the cutoff trench. Contact grout pipes were included in the wall placements and later grouted to refusal. The concrete walls then served as the tie-in for the embankment and grout curtain at each abutment, thus completing seepage control for the dam cutoff structure. (Reference Plate XI-4.)

10-05. To further inspect the effects of solutioning and to remove undesirable material, the cutoff trench was extended to station 14+10 on the right abutment. This additional portion of the cutoff trench was excavated 5 feet below the top of the existing sandstone bench. This shallow portion of the cutoff trench extended from station 12+75 to 14+10 where the abutment slopes steepened abruptly. No unusual conditions were discovered and no additional treatment, other than extending the downstream reverse filter, was required in this shallow portion of the cutoff trench.

10-06. To prevent transverse piping of the embankment into the adjacent foundation, openings in the upstream and downstream cutoff trench walls were washed to a depth of one third the width and treated with dental concrete where possible. Local areas of major deterioration and solutioning were covered with concrete wall sections constructed behind simple formwork. A double reverse filter was placed between the downstream contact of the impervious core and the treated limestone. This 10-foot reverse filter consisted of 5 feet of compacted Ind. #4 crushed limestone adjacent to the rock and 5 feet of compacted Ind. #14-1 natural sand between the crushed stone and the impervious core. This sand portion of the filter was lapped 2 feet over the stone at the top of rock. (Reference Figure X-1.) An additional filter was installed between the impervious core and the upstream wall of the trench. This upstream filter consisted solely of compacted Ind. #4 crushed stone. The purpose of this upstream filter was twofold. First, to accumulate the freewater seeping through the limestone and secondly, to fill the voids and overhangs where the impervious material could not be properly compacted. To remove the freewater from the excavation, slotted casings were installed at strategic points along the trench in the crushed stone portions of the filters. Submersible pumps were installed and the excess water was pumped out of the excavation. As the embankment reached the top of the casings, the pumps were removed, the casings were partially filled with crushed stone, and then capped with 3 feet of concrete. The downstream double reverse filter was installed the length of the trench, station 7+0 to station 14+10. However, the single upstream filter was installed only along the deep portion of the cutoff trench, station 7+0 to 12+50. A small monoclinal structure of shale was present between stations 8+0 and 8+40. In this area the Beech Creek Limestone was missing and filters were not required.

10-07. The Big Clifty Sandstone on the right abutment did not require additional foundation treatment other than removal of small overhangs. The sandstone on the left abutment, however, did require additional treatment. When first exposed, numerous thin uncemented dark sand seams up to 1.0 foot thick were evident. This portion of the



PATOKA LAKE
CUT-OFF TRENCH
DAM

FIGURE X-1

TABLE X-1

DAM AND DIKE
SAND AND GRAVEL FILTERS

Sieve Size	% Passing
Indiana #14-1 Sand	
3/8 inch	100
4	95 - 100
8	80 - 95
30	20 - 50
50	5 - 20
100	0 - 5
200	0 - 3
Indiana #4 Gravel	
1.5 inch	100
1 inch	70 - 90
3/4 inch	45 - 65
1/2 inch	10 - 30
3/8 inch	0 - 15
4	0 - 5
200	0 - 2

abutment was laid back to form a positive slope and to confine the solution zone treatments. When final excavation of the cutoff trench on the left abutment was complete, the soft seams were still evident on the final 1 to 1 slope face in sandstone. These soft seams did not leak appreciably during grouting and required minimal treatment. Overhangs were broken off and these soft seams were cleaned and packed with lean concrete. The Big Clifty Sandstone also formed the foundation for the Stage I cofferdam. In this area, no treatment other than chipping of overhangs was required. The Stage I cofferdam foundation was washed clean before the placement of embankment material.

10-08. The shale portion of the Golconda Formation required extra treatment on the right abutment. However, little extra treatment was required on the left abutment shale. The shale portion on the right abutment was exposed during outlet works excavation and formed a portion of that foundation. The right abutment portion of shale was left exposed during the interval between outlet works and dam construction. Consequently, some deterioration and weathering occurred. During initial grouting under the conduit, the poor condition of the rock near the surface was noted from numerous surface grout leaks in the immediate area riverward of the conduit. This section of shale further deteriorated between separate phases of the grouting operations. Great care was exercised during later grouting phases, by the use of packers, to pinpoint the extent and depth of weathering. Thus outlined, the deteriorated rock was removed in the impervious core area immediately before embankment placement. The rock immediately under the riverside section of the concrete conduit plug was found to be soft and weathered. A small portion of this rock was removed and replaced with additional lean concrete.

10-09. The portion of shale on the left abutment required only normal cleanup and scaling before embankment placement. The variation in treatment of the two abutment portions of the Golconda Shale was due to the variations in lengths of exposure. The left abutment portion of this shale was protected against deterioration until excavated shortly before embankment construction. The portion of this shale on the right abutment, however, was exposed for a long interval between construction contracts and was used as a portion of the haul road during dam construction. The Golconda Shale on both abutments was cleaned by air pressure and dampened immediately before placing the embankment.

10-10. During design, the threat of embankment materials piping through the Golconda Limestone was realized. Consequently, during the outlet works construction contract, provisions were made to uncover both limestones for mapping and further study. During the outlet works construction, the Golconda Limestone on the right abutment was excavated to the point where a treatable face was exposed. At this point the face of the limestone presented many small clay-filled cavities and solutioned joints near the basal contact. The uncovering of the Golconda Limestone on the left abutment during outlet works construction revealed extensive solutioning with a resultant blocky structure.

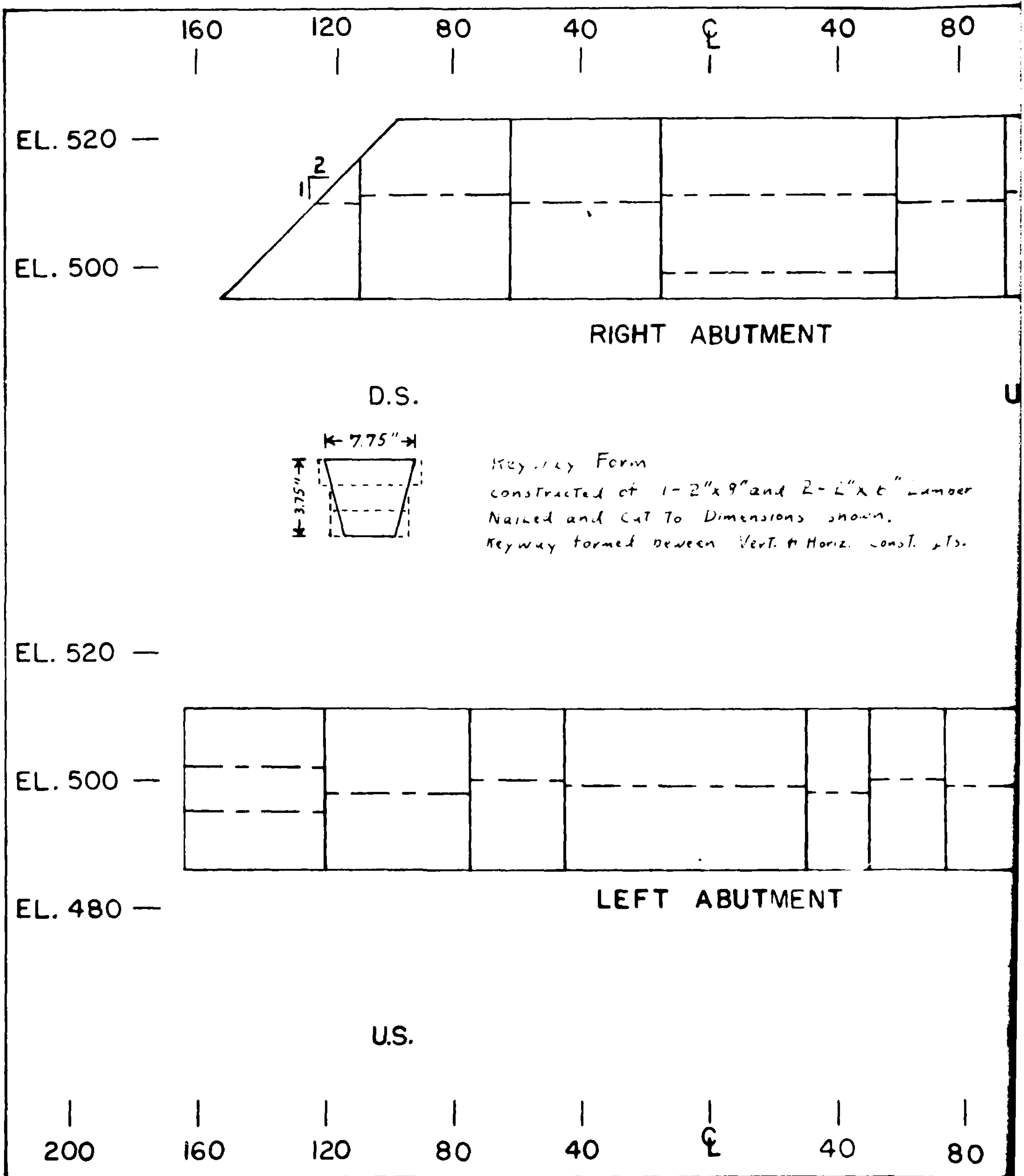
10-11. For the dam and spillway contract, 12-inch continuous concrete walls over the exposed limestones on the abutments were designed. Construction of the concrete wall and treatment of the Golconda Limestone on the right abutment proceeded according to plans since this limestone was already excavated to its final configuration. The small portion of the 12-inch wall under the Stage I cofferdam area was placed first without problems. All clay-filled cavities and joints were cleaned to a depth corresponding to three times the width, and filled with concrete as part of the wall placement. Loose rock was scaled off and contact grout pipes were installed prior to concrete placement. For this short section of wall adjacent to the tower, only vertical construction joints were utilized. Upon completion of the wall and after form stripping, the contact holes were grouted to refusal using minimal pressures. During initial cleanup, one open void was found in the limestone above and behind a clay-filled cavity. Extra grout pipes were installed in this cavity and it was satisfactorily grouted during contact grouting operations.

10-12. Before construction of the remaining portion of the right abutment wall, curtain grouting in the area was completed. Upon completion of grouting operations, cleaning of the clay-filled cavities was performed and the remaining portions of the wall were constructed. The 12-inch wall was built in such a manner that construction joints were kept to a minimum. Where construction joints were needed, keyways between wall sections were included (see Figure X-2 for construction joint and keyway details). However, no waterstops between joints were utilized in this wall. The only reinforcing, beside form ties, was provided by a 6x6 inch, #4 gauge mesh in the impervious core area and extending 6 feet beyond the upstream and downstream core limits. A total of 153 contact grout pipes were included in the right abutment wall and were grouted to refusal upon completion of the wall. The outer wall limits conformed to the original embankment slopes, 3 to 1 upstream and 2 to 1 downstream.

10-13. Treatment and construction of the concrete wall at the Golconda Limestone on the left abutment of the dam proved to be more difficult. The original contract required presplitting this limestone on a 1 to 1 backslope a short distance behind the precontract exposed face. When the limestone was excavated to this point, the extent of solutioning required further excavation. The contract was modified and the Golconda Limestone was further excavated to station 3+65 (toe of backslope). The backslope was changed to 1 horizontal to 2 vertical.

10-14. Upon completion of this revised excavation, extensive solutioning was still evident near the base of the Golconda Limestone. The contract was further modified to move the excavation another 30 feet back to 3+23. Upon completion of this changed excavation, extensive solutioning was still present near the base of the limestone. However, further excavation was not feasible and the contractor was instructed to build the 12-inch concrete wall at this point.

10-15. Curtain grouting was commenced, but had to be stopped immediately due to the safety hazards presented by the large overhanging


160
120
80
40
℄
40
80
EL. 520
EL. 500
1
2
RIGHT ABUTMENT
D.S.
U
7.75"
3.75"
Keyway Form
constructed of 1-2"x9" and 2-2"x6" Lumber
Nailed and Cut To Dimensions shown.
Keyway formed between Vert. & Horiz. const. jts.
EL. 520
EL. 500
EL. 480
LEFT ABUTMENT
U.S.
200
160
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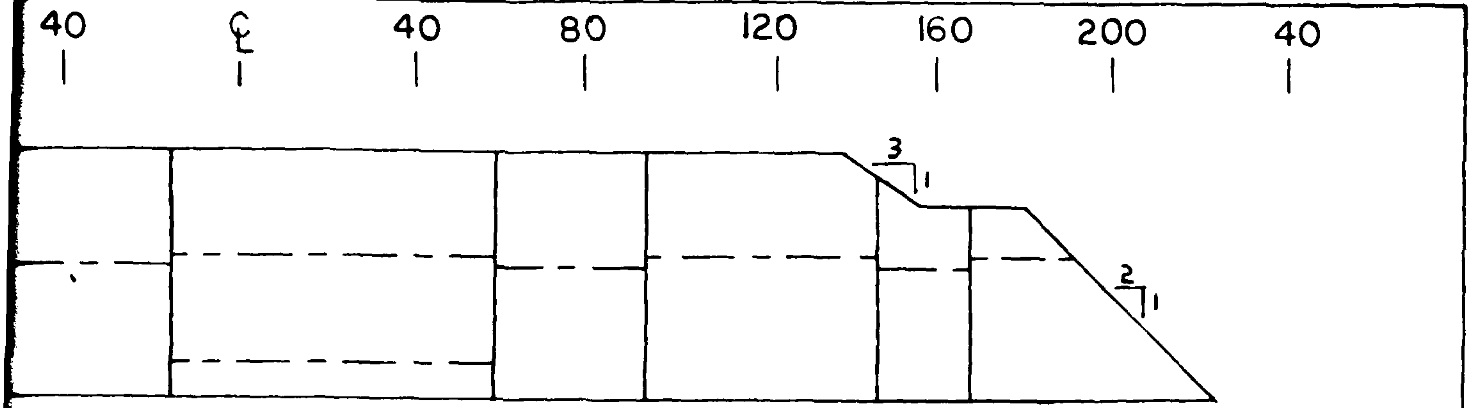
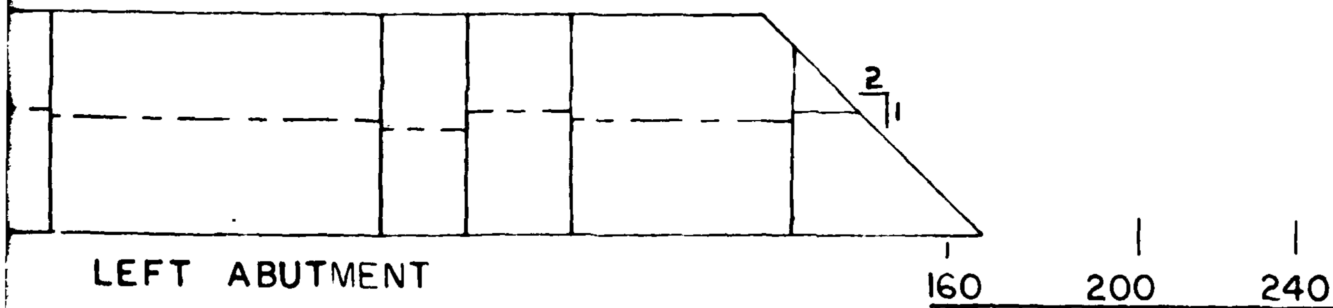
40
80



RIGHT ABUTMENT

U.S.

...ey Form
...ructed of 1-2"x8" and 2-2"x6" Lumber
...d and Cut To Dimensions shown.
...y formed between Vert. & Horiz. Const. Jts.



LEFT ABUTMENT

D.S.

40 ℄ 40 80 120

PATOKA LAKE

CONSTRUCTION JTS.
IN CONCRETE WALLS
GOLCONDA LIMSTNE

FIGURE X-2

cavities. The contractor was then instructed to place dental concrete in the cavities before grouting. These cavities were cleaned out to a depth of three times their width when possible, contact grout pipes were installed, the faces were formed, and the cavities were filled with lean concrete. Upon completion of the dental concrete, curtain grouting was restarted. When curtain grouting was completed back away from the limestone face, wall construction was commenced. The 12-inch wall was constructed in a manner similar to the wall over the limestone on the right abutment of the dam.

10-16. Final left abutment Golconda Limestone excavation limits formed a notch with angled wingwalls upstream and downstream. The 12-inch wall was warped to conform to these excavation limits. The original downstream 2:1 embankment slope line was used to determine the configuration of the downstream portion of the wall. The upstream portion of the wall extended to a point 144 feet right of the centerline at station 3+70, which was the upstream limits of the limestone exposure. Contact grout pipes were preset through the wall and grouted to refusal with minimum pressures upon completion of the wall. A total of 241 contact grout pipes were installed in the dental and wall construction against the left abutment Golconda Limestone.

10-17. The next rock in ascending order requiring treatment was the Hardensburg Shale. The extremely poor cementation of the shale led to grouting and treatment difficulties on the right abutment. During initial grouting, casings set in this material would not hold the stand drills and grouting of this rock was postponed until completion of the concrete wall over the lower limestone. On the second grouting attempt, packers were utilized to isolate the highly weathered areas to determine removal limits. Upon completion of grouting, the unstable rock was removed within the impervious core limits to form a suitable foundation. Again, as with the lower Golconda Shale, the length of exposure had an adverse effect on the deterioration of the rock.

10-18. The Hardensburg Shale on the left abutment was exposed in a fresh cut and did not require abnormal treatment. Some difficulties were encountered during later stages of grouting, but normal cleanup was the extent of treatment in this portion of the shale. The Hardensburg Shale on both abutments was cleaned with compressed air and wetted immediately before placing the impervious portion of the embankment. The thin 2- to 4-foot sandstone bed near the top of this shale on the left abutment was found to be highly jointed. During cleanup of this bed, the joints were found to be open and removal of this bed within the impervious core was deemed necessary.

10-19. The next rock unit in ascending order requiring treatment in the dam foundation was the Glen Dean Limestone. This formation was above the foundation limits on the right abutment and consequently did not require treatment. On the left abutment, however, this formation was inside the foundation and required extensive treatment.

10-20. As part of the outlet works contract, the Glen Dean and Golconda Limestones were both uncovered and mapped on the left abutment of the dam. The mapping details were included as part of the dam contract drawings. The original contract required presplitting the upper limestone at station 2+40 (top of slope), with angled wingwalls beyond 60 feet upstream and downstream of the centerline. During initial dam excavation on the left abutment, solutioning in the Glen Dean Limestone was found to extend much further into the abutment. Further exploratory work in the form of 5 each air-trac holes and additional cleaning was performed in the Glen Dean Limestone to determine the extent of solutioning. As a result of this exploratory work, the toe of the backslope in the Glen Dean Limestone was moved back to station 2+30. The angle of the backslope was also steepened from a 1 to 1 slope to a 1 to 2 slope. In excavating this modified backslope, deeper wingwalls were formed upstream and downstream of the new face.

10-21. Upon completion of the revised excavation in the Glen Dean Limestone, some solutioning still existed. Notably one large vertical solutioned joint near the centerline and an intersecting solutioned horizontal bedding plane near the base of the limestone. The vertical joint formed a portion of the upstream wall. At this point, further excavation was not feasible and the decision was made to install the 1-foot concrete wall at this location. Grouting prior to wall placement was commenced and numerous problems were encountered with the intersecting solutioned planes as discussed in the grouting section of this report. Grouting problems and communications between different sections of the limestone face led to the decision to install additional grout holes in the concrete wall and grout upon completion. Additional treatment before wall placement was limited to cleaning and scaling of loose rock. All cavities were cleaned and filled with the wall placement. This wall was constructed in a similar manner to the lower concrete walls covering the final excavated limestone exposures within the core trench.

10-22. The wingwalls formed in the Glen Dean Limestone during excavation were not faced with a 1-foot concrete wall. Instead, the cavities in the upstream wingwall were filled with Ind. #4 crushed limestone and the cavities in the downstream wingwall were filled with lean dental concrete after cleaning. Consequently, the main portion of the concrete wall was reduced to extend 60 feet upstream and 40 feet downstream of the dam centerline. Stage grouting of the additional grout holes was performed after the wall was stripped. As with other concrete walls, a total of 41 contact grout pipes were installed in the wall and grouted after stripping. The top of the concrete wall was located at elevation 561.5. At this elevation, the Type "B" accelerograph shelter footer was formed on the dam centerline. The thin 3 to 5-foot thickness of Glen Dean Shale overlying the limestone did not require treating other than normal cleanup.

10-23. The concrete walls placed over the face of the limestones exposed in the dam foundation offered a number of advantages for treating the solutioned conditions. First, cavities were either filled or blocked allowing a reasonable condition for treatment by grouting.

Secondly, the abutment tie-in for fill placement presented ideal conditions for maximum compaction of the adjacent embankment. Thirdly, this concept could be used where rock conditions prevent a reasonable construction approach to dental treatment.

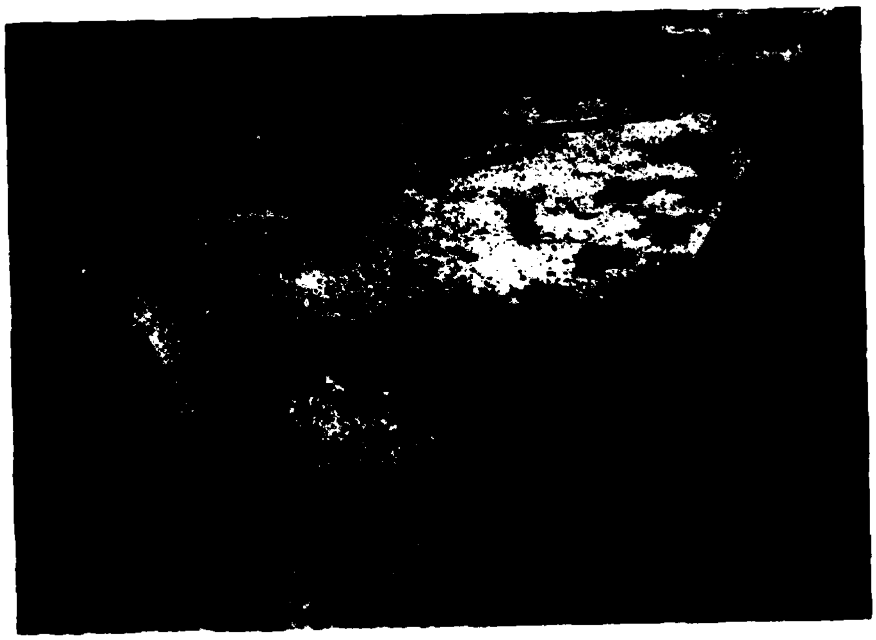
10-24. The Mansfield Sandstone was not present in the dam foundation. The only dam exposure of this rock was found at the top of the left abutment, above the embankment. Consequently no treatment of this rock, other than scaling, was required for this rock in the dam area.

10-25. In summary, dam foundation treatment concentrated on treating the varying rock types to prevent piping of the completed embankment. Consequently, any defect in rock surfaces through which embankment might pipe was singled out for extensive treatment. Large and continuous openings in the various rocks were treated by dental concrete after cleaning and by reverse multicomponent filters, depending upon their location within the foundation limits. Smaller openings were treated either by filters or by limited dental concrete. Generally, treatment against the downstream portion of the impervious core was much more extensive than the treatment upstream of the impervious core. Many of the foundation conditions encountered were not expected and required immediate remedial treatment. As a result of these ongoing design changes, extensive modifications were made to the original contract. As an additional consequence, many of these revised and added drawings were not added to the contract drawings. To remedy this shortcoming, a conscientious effort was made to include these changes in the "As Built" drawings, embankment and foundation reports.

17 June 1976
Dam Core Trench Sta 12+50. Completed concrete wall over Beech Creek Limestone face.

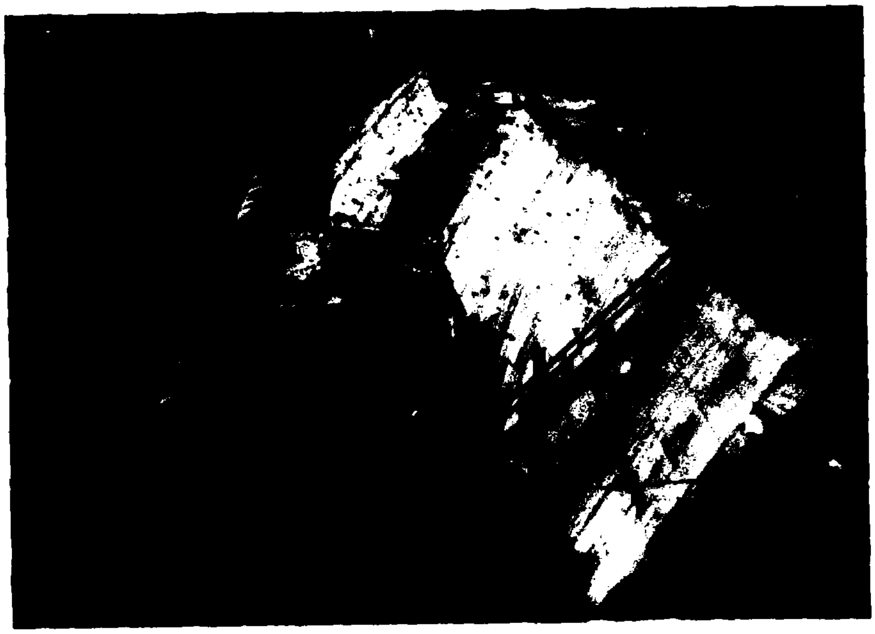
1 October 1976
Dam Left Abutment, Golcónda Limestone. Downstream end of 1 ft. concrete wall. Note: 2:1 slope.

<u>3 December 1976</u>
Dam Left Abutment, Golconda Limestone. Placing last portion of 1 ft. concrete wall.



<u>15 July 1976</u>
Dam Right Abutment, Golconda Limestone. First lift of concrete wall near centerline. Note: wire mesh and keyways in construction joints.

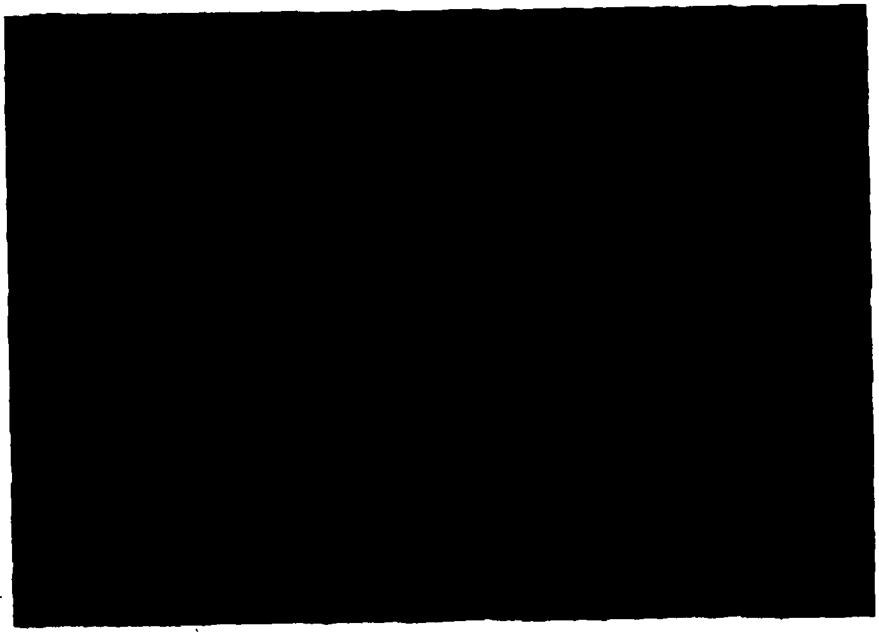
Plate X - 2.



10 August 1976
Dam Right Abutment. Completed concrete wall over Golconda Limestone. Note: uncovered li stone at base of wall.

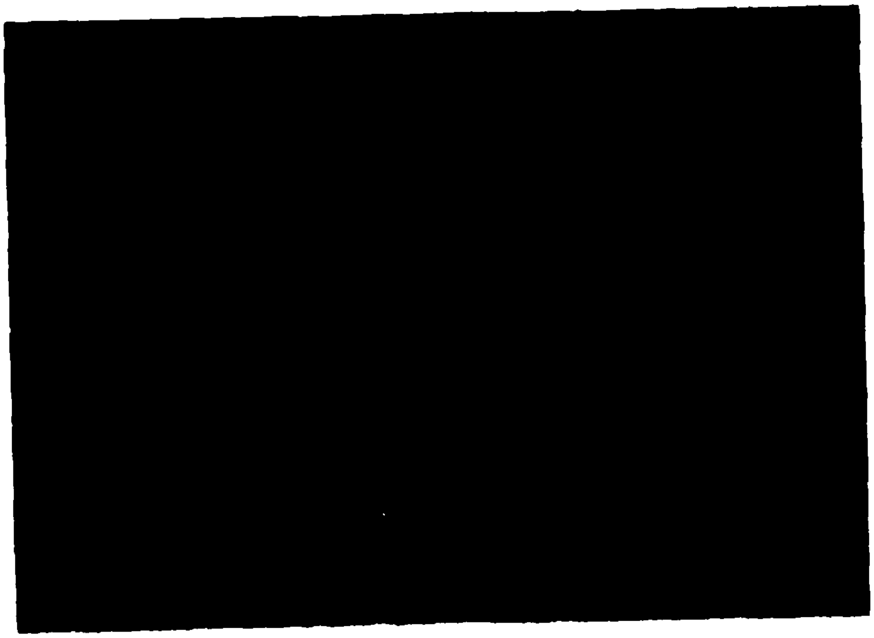
20 July 1976
Dam Core Trench, Sta. 7+50. Local water control and filter zone details. Dental concrete will be placed when fill reaches the base of the LS.

Plate X - 3.



18 July 1977

Dam Left Abutment at Base of Glen Dean Limestone. Jointing pattern in top SS bed of Hardinsburg Formation. Note: limestone wing wall condition.



18 July 1977

View Upstation Along Dam Centerline Showing stained jointing in SS. This SS was later removed in core area. Note: dashed red imprevious core outline.

Plate X - 4.

XI DIKE FOUNDATION TREATMENT

11-01. Of the various means of dike foundation treatment, most centered around the highly solutioned limestone at both abutments. Foundation treatments at the left abutment of the dike has been discussed briefly in the special grouting report. Discussion of that area will, therefore, be limited in this report.

11-02. The primary means of treating of the Glen Dean Limestone in the dike foundation took the form of a positive cutoff extending into both abutments to a point where solutioning ceased to be a problem. The left abutment cutoff accomplished this end, but the cutoff into the right abutment did not reach a satisfactory end point. The left abutment cutoff was added as a modification to the original contract. The positive cutoff in the right abutment was part of the original contract, and had to be modified by widening and by additional treatment at the end wall. This was required because of the continued solutioning beyond the end of the embankment. The positive cutoff completely removed the solutioned limestone down to impervious shale for a sufficient width to install the dike embankment.

11-03. During initial excavation of the right abutment cutoff trench, the full extent of solutioning and pinnacle formation became apparent. To protect the embankment from piping through the solutioned and partially filled cavities, the cutoff was widened to provide room for additional treatment in the form of reverse filters. In the fullest section of limestone, stations 12+50 to 18+0, the cutoff was widened from 15 to 35 feet. The 35-foot width was selected to include the 15-foot wide original impervious core, a 10-foot double zoned reversed filter against the downstream wall, and an additional 10-foot wide rock transition onto the upstream side of the impervious core. Limestone side slopes were vertical for this stretch of the cutoff trench.

11-04. The cutoff trench width in the thinner section of limestone, stations 8+0 to 12+50, was increased to 53 feet at the base of the excavation. This extra width beyond the 35 feet was filled by extra transition (9-foot) material upstream and impervious (9-foot) core material downstream. Limestone side slopes were 1 horizontal to 2 vertical for this stretch of the cutoff trench.

11-05. The 10-foot wide downstream reverse filter consisted of an outer 5-foot wide compacted gravel and a 5-foot inner zone of compacted sand. The gravel zone consisted of Ind. #4 crushed limestone compacted with four passes of a plate vibrator. The gravel zone extended up from the base of the excavation to the top of the limestone. The inner zone consisted of Ind. #14-1 natural sand compacted to 80 percent of maximum relative density with a plate vibrator. The sand portion of the filter extended from the base of the excavation to the top of the gravel filter, then lapped 2 feet over the top of the gravel filter and exposed limestone. Prior to placement of this filter on the downstream wall of the excavation, the clay-filled cavities in the limestone were cleaned to a depth equal to three times their width. Cleaning was performed by hand and water blasting. The cleaned-out void portions of the cavities were then backfilled with formed and vibrated dental concrete. This treatment was performed on the downstream face of both right and left dike abutment cutoffs. During treatment, one overhang on the downstream right abutment wall was inadvertently sealed off leaving a large void behind the dental concrete. An air-trac was later used to drill holes into the resultant void which was then filled with concrete, gravity fed through the drilled holes.

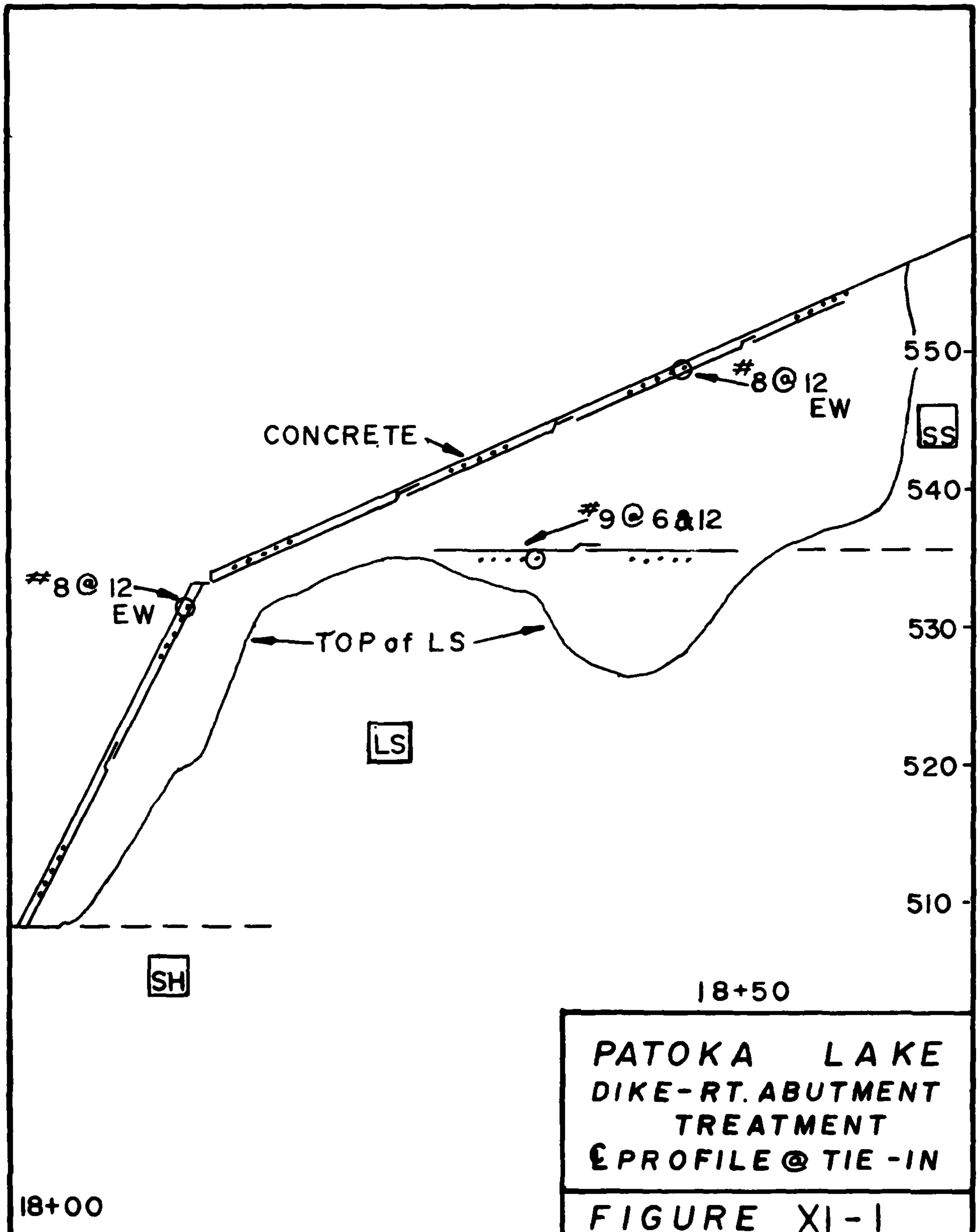
11-06. The upstream face of the limestone cutoff was treated differently. In addition, the upstream portion of the cutoff was treated slightly differently at the right abutment than at the left abutment of the dike. The right abutment upstream face of the limestone excavation was treated basically with Ind. #4 gravel. Clay-filled cavities within the limestone were not specifically dental treated with concrete as was done on the downstream face. The cavities, instead, were filled with Ind. #4 gravel and compacted along with the transition material. Any overhang or cavity that was too large to allow compaction was filled with dental concrete. One large overhang in particular between stations 17+15 and 17+60 was treated in this manner. Instead of forming across the face of this one overhang, gravel was placed in the opening and concrete was utilized to fill the remaining void behind the gravel. The upstream limestsone face in the left abutment cutoff excavation was treated with Ind. #4 gravel only. No dental concrete was used on this wall.

11-07. The limestone portion of the end wall on the left abutment did not require extensive treatment. Loose rock was scaled off and the small joints and overhangs were filled with hand-placed dental concrete. The shale at the base of the dike cutoff on both abutments did not require extensive treatment. Soft, weathered pockets in the shale were removed and the shale was air cleaned and wetted before placing the impervious fill. Treatment of the overlying sandstone was limited to scaling of loose rock and chipping or packing concrete in overhangs. The highly irregular sandstone at the limestone contact on the left abutment required some upward extension of the gravel filter.

11-08. The surface of the limestone left in place, station 0+35 to 0+76 on the left abutment of the dike, was irregular with numerous small potholes and depressions. Before impervious fill placement, the surface of the limestone was washed clean and all depressions were filled with lean concrete to provide a uniform, smooth surface for fill placement. Any open core hole observed in the foundation was washed clean and filled with lean concrete.

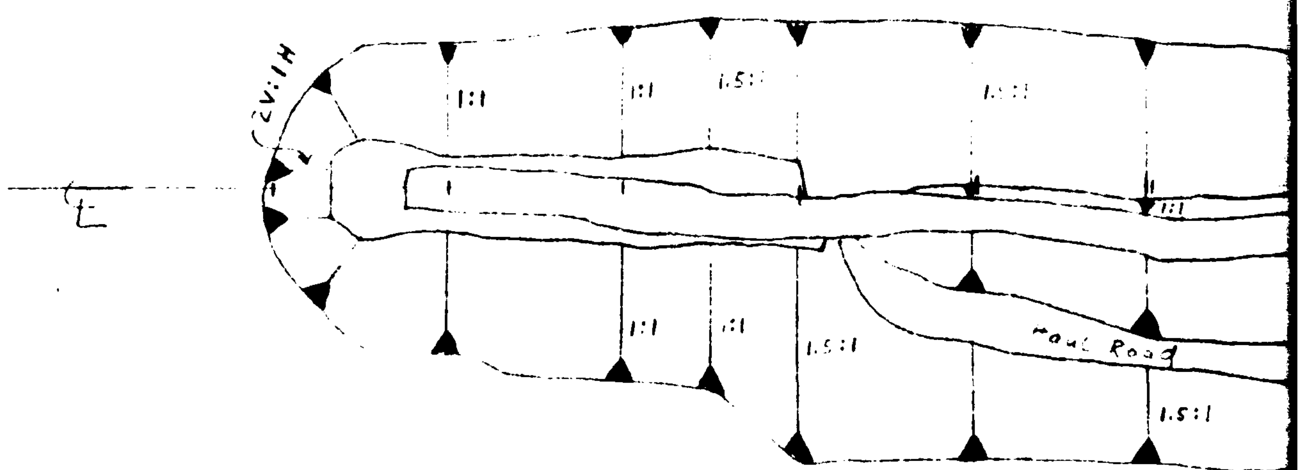
11-09. The end wall on the right abutment portion of the positive cutoff excavation required the most extensive dental treatment in the dike. As noted in the excavation section, this wall was found to be in extremely poor condition. Two large solutioned joints were found nearly paralleling the centerline on the upstream and downstream corners of the excavation. In addition, several large clay-filled cavities were noted adjacent to the solutioned joints. The top of the limestone was solutioned with a large depression formed behind the slope face. The overlying sandstone had collapsed into this solutioned depression forming a highly unstable collapse structure.

11-10. Treatment of this area took the form of constructing a 4-foot minimum thick reinforced concrete wall over the irregular limestone face. During placement of this wall, all irregularities were cleaned and filled with concrete to the top of limestone. All cavities near the face were washed clean to a depth of three times their width and refilled with concrete as part of the first wall lift. In addition, the deep vertical solutioned joint upstream of the centerline was washed clean and filled with concrete as part of the first lift. The original backslope in the sandstone was reconstructed by replacing the cleaned out portion of the collapsed area with concrete. A re-steel mat was placed over the limestone face for crack control. One mat consisting of #8 rebar at 12-inch centers was placed over the slope faces. A second mat consistsing of #9 rebar was placed horizontally at the top of the limestone. The wall was placed in three sections. The first section was placed to elevation 532 which was 2 feet above the top of the limestone. Due to safety constraints, the back of the sandstone collapse zone could not be cleaned to specification standards. Grout pipes were installed through the concrete wall for grouting. A total of 122 contact grout pipes were installed with the first concrete placement. In addition, six vertical grout pipes were installed in the first lift for drilling and grouting of the large solutioned joint upstream of the centerline. An additional 93 contact grout pipes were set in the second concrete wall placement. Further casings were set in the second concrete placement for curtain grouting. All holes were grouted before embankment placement. The end result was a reconstructed "Tie-in" block for the dike embankment.

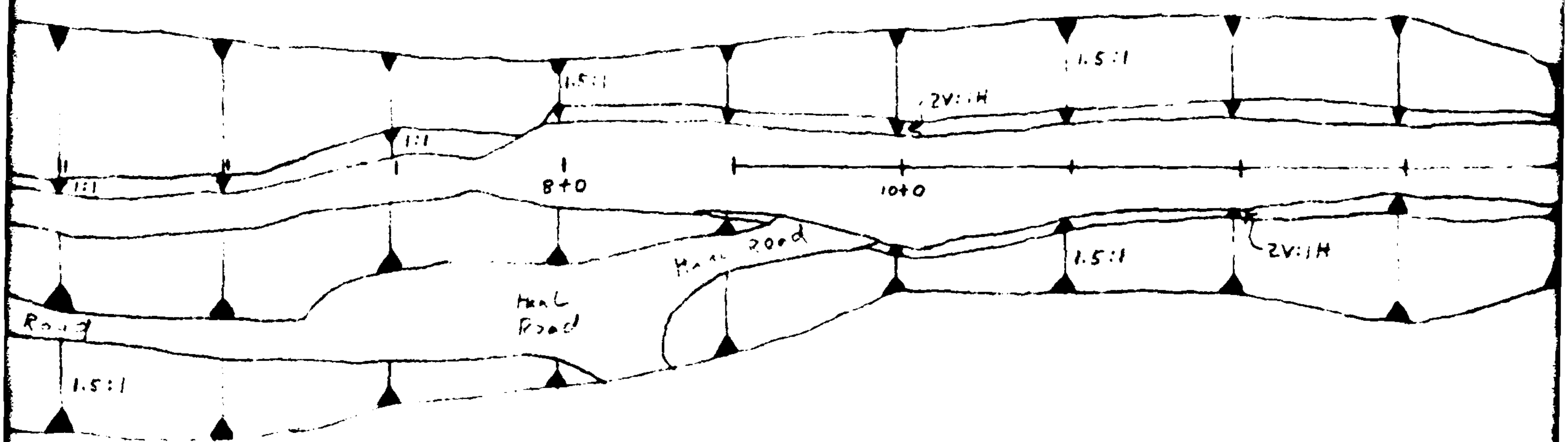


PATOKA LAKE
DIKE-RT. ABUTMENT TREATMENT
℄ PROFILE @ TIE-IN

FIGURE XI-1



2V:1H
1:1
1:1
1.5:1
1.5:1
1:1
1:1
1:1
1.5:1
1.5:1
Haul Road



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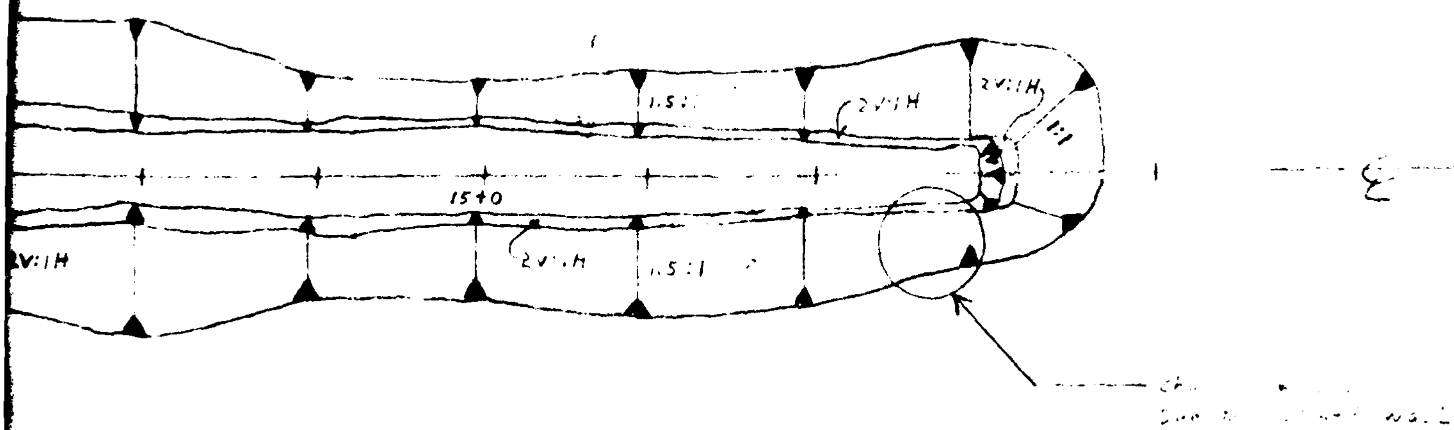

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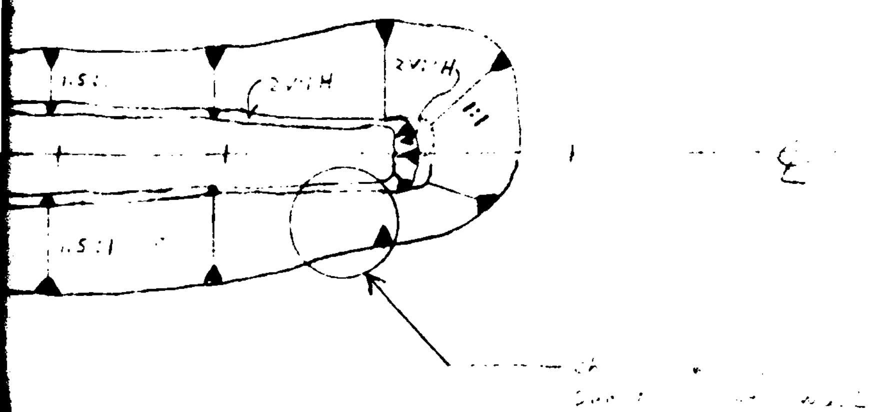


PATOKA LAKE

PLAN VIEW
DIKE EXCAVATION
AS BUILT

FIGURE XI-3

3

4

9 July 1977
Dike Right Abutment Collapse Structure at Sandstone-Limestone Contact. Note: yellow flagged grout curtain limits.

17 August 1977
Dike, Right Abutment. Second lift concrete wall forming. Note installation of grout casings.

Plate XI - 1.

XII POSSIBLE FUTURE PROBLEMS

12-01. Future problems with the foundation on this project should be fewer than normal due to the extensive treatment and foundation revisions that occurred during construction. Serviceability over the long life of the project was one of the prime concerns for selecting most of the modifications. However, close observation of the modified foundation is necessary to insure that serviceability.

12-02. The primary observations and potential problem areas center around the grout curtain. Although great care was exercised in constructing the grout curtain, the long-term reliability of grouting in highly solutioned limestones is not completely assured. Therefore, continued close monitoring of the wellpoints downstream of the grout lines is mandatory. Additional wellpoints may be necessary if present conditions change.

12-03. As noted previously, the grout curtain on the right abutment of the dam was terminated at station 177+45. During initial stage exploratory drilling up the hill beyond this point, a large tool drop was experienced in one of the bore holes. Consequently, monitoring of this area will be required throughout the service life of the project to determine the need for additional grouting. Also related to this area is the short seepage path to Georges Creek. This short seepage path, combined with the structure of the limestone, creates a potential for pool loss into this adjoining drainage basin. This area was monitored during pool filling, but further periodic monitoring of this area will be necessary.

12-04. A portion of the grout line on the right abutment near the shop and office building is aligned parallel to one of the primary joint patterns in the Golconda Limestone. This situation tends to reduce the effectiveness of the grouting program. Consequently, the potential for seepage exists in this area, and should be monitored closely. As an aid to monitoring, wellpoints 15 through 20 were installed on the right abutment 50 feet downstream of the grout curtain. All of these wellpoints, along with the spring at the stilling basin, react quickly to periods of intense rainfall. If any changes occur in these wellpoints that are not related to rainfall, reduced effectiveness of the grout curtain should be suspected.

12-05. The wellpoint (WP 12A) installed in the dam turnaround was specifically installed to monitor stability of the grout curtain. This wellpoint was installed in a clay-filled solutioned seam within the Golconda Limestone. As discussed previously in this report, this clay material is extremely weak and subject to piping. Grouting was very thorough in this area during construction, but the stability of the clay fillings may be reduced during pool filling and subsequent pool level fluctuations. Consequently, this wellpoint needs to be monitored closely to detect early signs of instability in the clay-filled cavities within the limestone.

12-06. As discussed previously in this report, the area between the dam and spillway was not grouted. Provisions were made to monitor this area during and after pool filling to determine if future grouting would be necessary. Solutioned areas within the upper limestone are known to exist and continued close monitoring of the area is necessary.

12-07. The grout curtain on the left abutment of the dike was terminated at a point convenient for contractual requirements. Numerous water losses and drilling difficulties were encountered landward of the terminal grouting point during exploratory drilling. In addition, the limestones dip into this abutment allowing easier migration of impounded water. No piezometers were installed landward of the grout curtain to allow direct monitoring of this area. As a result, this portion of the left abutment of the dike needs to be monitored to detect any signs of seepage.

12-08. During the early stages of dam construction, the limestone knob (borrow area 3A) upstream of the dam on the left abutment was used by the contractor to construct haul roads and equipment working pads. Upon completion, this area was covered with waste material removed during dam excavation. The purpose of this cover was to inhibit the seepage of water through the joints in the limestone, downstream between the dam and the spillway. During late stages of the contract, several uncovered open solutioned joints were noted in the remaining limestone. Apparently the covering material had collapsed into the openings and had been washed away during pool fluctuations. A potential seepage path thus exists to the ungrouted area between the dam and the spillway.

12-09. During dam construction, the slope of the downstream face was changed from a 2 to 1 slope to a 3 to 1 slope to facilitate maintenance. This change in slope was constructed by overlaying the original slope with waste material. The majority of the embankment was already completed when this modification was made. As a result, the concrete wall facings over the limestones on both abutments were also completed. Consequently, the waste material in these areas was placed directly over unprotected solutioned limestone. The possibility of piping of this waste material exists, but should not be detrimental to the safety of the structure. The difficulty lies in determining exactly which material pipes. If this piping situation does arise, great care should be exercised in determining which material pipes and to determine if this action actually has any detrimental effect upon dam stability.

12-10. In this same light, the upper Glen Dean Limestone on the left abutment of the dam was not completely faced with a concrete wall. The upstream and downstream wingwalls created in the limestone during excavation were not faced. These wingwalls were highly solutioned and eroded at the time of exposure. Both these areas were treated by filling the voids with crushed limestone before placing the embankment. However, a portion of the embankment has been placed against the exposed rock and a potential for piping does exist.

12-11. The sandstone exposed above elevation 564 on the left abutment of the dam was left untreated. During the modified rock excavation procedures on the left abutment of the dam, a steepening of the final presplit line was required in this sandstone. Over the last several winters, this rock has continued to spall. The resultant ditch filling by spalled rock does not present a safety or structural problem, but will continue to be a maintenance problem throughout the life of the project.

12-12. The spillway floor presents a potential for detrimental erosion. As discussed previously, many difficulties in grading the spillway floor were encountered. As a result, much of the spillway floor downstream of the sill is covered with loose sand and rock fragaments. If the spillway should ever be used for its intended purpose, most of this loose material would be removed and the underlying soft sandstone floor would be subjected to channelization and resulting erosion. Further complicating the erosional problems would be the numerous rubble-filled collapsed structures encountered in the spillway floor. If water movement occurs through these structures, deep seated internal erosion of the clay-filled solutioned joints in the limestone may occur. If these situations occur, paving a portion of the spillway floor may be required.

12-13. Since construction, several of the completed paved ditches have been undermined by runoff water. Most serious is the 7-foot bottom paved ditch at the outlet end of the spillway. This ditch needs to be repaired and continual maintenance will be required due to the lack of sodding and the absence of lugs in the trench. Other ditches on the left and right abutments of the dam are showing signs of undermining which need to be corrected and monitored.

12-14. During construction of the south access road a very slickensided "fire-clay" shale fed by seep water was encountered between stations 207 and 218 approximately. Almost immediately, the side hill slope in this area began to slide. This sliding has continued thorugh the dam contract and will continue until the proper methods of treatment are utilized to stabilize this area.

XIII CONTRACT CHANGES AFFECTING THE FOUNDATION

13-01. As part of the documentation of the foundation, Tables XIII-1 and XIII-2 are included. Table XIII-1 is a record of design changes made during construction. Table XIII-2 is a list of all the contract modifications affecting the foundation. These tables are related in that the decisions and changes listed in Table XIII-1 resulted in most of the contract modifications listed in Table XIII-2. The contract modifications listed in Table XIII-2 were primarily the means of paying for the design changes. Many of these modifications were finalized after the fact. For these reasons, the two tables have not been combined. This data is included as a reference to facilitate the investigation of conditions if a foundation problem should arise in the future.

TABLE XIII-1

RECORD OF DESIGN
EVALUATION AND CHANGES
DURING CONSTRUCTION

AREA	DESCRIPTION OF CONTRACT WORK	DATE	ORGANIZATION UNIT REVIEW	COMMENTS
Dam	Excavation of Dam Foundation	1 Dec 75	ORLED	Reduce impervious core width of dam to 24 feet with vertical slopes, thus reducing dam foundation cutoff to 24 feet from mar of 60 feet. Revise contract drawings.
Dam	Excavation of Dam Foundation	9 Feb 76	ORLED	Approval to use 100,000 yards of rock to provide accessibility in waste areas. Rock to be obtained by additional rock excavation of right side of spillway. This is to be accomplished at no additional cost to the Government.
Dam	Spacing of Grout Holes Right Bank of Dam	8 Dec 75	ORLED	Concurred with drilling and grouting on .25 foot centers on right abutment of dam to bottom of Golconda limestone on centerline "B" holes.
Dam	Excavation of Dam Foundation	12 Mar 76	ORD, ORLED, ORLCD	Due to set back of design slope in Glen Dean limestone, to avoid a cut extending above the Glen Dean limestone on the dam left abutment, it was agreed that the abutment cuts could be made on slopes of 2V to 1H instead of 1V to 1H as called for in contract plans.
Dam	Treatment of Lower Dam Foundation	12 Mar 76	ORD, ORLED, ORLCD	When valley bottom is cleaned up enough to expose all the formations, the District is to notify ORD for a site inspection and ORD is to work with ORL on appropriate foundation treatment.
Dam	Excavation of Lower Dam Foundation	20 Apr 76	ORD, ORLED, ORLCD	Remaining portion of solutioned Beech Creek limestone to be removed by cutoff trench in valley bottom station 7+18 to 12+50. Cutoff trench to extend 14 feet U.S. of centerline and 22 feet D.S. of centerline.

TABLE XIII-1

RECORD OF DESIGN
(Continued)

AREA	DESCRIPTION OF CONTRACT WORK	DATE	ORGANIZATION UNIT REVIEW	COMMENTS
				Any sizeable openings in the limestone on the U.S. face will be sealed with dental concrete to prevent a run around past the stepup in the core trench at 12+50. Openings larger than about 1 inch on D.S. slope of core trench will be cleaned out and sealed with sand cement grout or concrete. The D.S. slope of the cutoff trench including the overlying Big Clifty sandstone will be covered by a 5-foot wide blanket of crushed stone ranging in size from 1-1/2" to Indiana No. 4 which in turn will have a 5-foot wide zone of concrete sand next to impervious core. These materials will be compacted with a vibratory roller. The leading edge of the Big Clifty sandstone contact with the Beech Creek limestone will be broken back U.S. and D.S. of cutoff trench to a stable slope and large openings filled with concrete and the remaining face of the contact area covered with two 5-foot layers of the above materials.
Dam	Drilling and Grouting Bottom Dam Area	20 Apr 76	ORD, ORLED, ORLCD	From Station 7+18 to Station 12+50, only a single line grout curtain 20 feet deep with primary holes on 20-foot centers will be installed as an exploratory line unless unfavorable conditions are found.
Dam	Foundation Excavation	20 Apr 76	ORD, ORLED, ORLCD	A 36-foot wide trench will be presplit down toe of left abutment through the Big Clifty sandstone and Beech Creek limestone with backslopes of one on one from approximate Station 6+75 to remove irregularities and weathered rock. The highly solutioned rock at the toe in the Beech Creek limestone will be covered by a concrete wall. Sand seams encountered in the Big Clifty sandstone will be dental treated to prevent piping. U.S. and D.S. of the core trench, the highly weathered and cavernous zone at base of Big Clifty sandstone will be covered with two five layers of commercial materials after sizeable cavities are blocked with concrete.

TABLE XIII-1

RECORD OF DESIGN
(Continued)

AREA	DESCRIPTION OF CONTRACT WORK	DATE	ORGANIZATION UNIT REVIEW	COMMENTS
Dam	Dam Foundation Grouting	20 Apr 76	ORD, ORLED, ORLCD	The second zone of grout curtain to be deepened to extend through the Beech Creek limestone from Station 3+50 to 7+18 to prevent leakage from developing through openings found in the Beech Creek limestone and Big Clifty sandstone contact area.
Dam	Excavation Left Abutment Dam	22 Jun 76	ORLED, ORLCD	Change left abutment slope in Golconda limestone. Move back 30 feet to Station 3+20 due to high number of solutioned cavities.
Dam	Excavation Left Abutment Dam	8 Mar 77	OCE, ORD, ORLED, ORLCD	The Glen Dean limestone slope as laid back is in a condition which can be treated with dental concrete and a concrete facing. The Golconda slope as laid back although solutioned and filled with clayey soil can be treated to protect the embankment by removing the soil at the face of the cavities and plugging the major cavities with concrete.
Dam	Grouting Left Abutment of Dam	8 Mar 77	ORD, ORLED, ORLCD	The triple line grout curtain from grout Station 141+00 ahead on dam left abutment will be completed according to contract.
Dam	Concrete Slab on Glen Dean Slope Left Abutment of Dam	19 May 77	ORD, ORLED, ORLCD	Place designed wall in contract against Glen Dean slope for 60 feet upstream of centerline and 40 feet downstream to top of dam. Fill upstream side slope overhangs with Indiana No. 4 stone and use dental concrete in cavities below top of embankment on D.S. side. Use reverse filters for small openings.

TABLE XIII-1

RECORD OF DESIGN
(Continued)

AREA	DESCRIPTION OF CONTRACT WORK	DATE	ORGANIZATION UNIT REVIEW	COMMENTS
Dam	Grouting Left Abutment Glen Dean Slope	19 May 77	ORD, ORLED, ORLCD	Final grouting of the Glen Dean slope "B" line can better be grouted after the concrete facing is placed over the end face of the excavation.
Dam	Underdrain from Spring Right Side of Stilling Basin	19 May 77	ORD, ORLED, ORLCD	Construct a 12-inch underdrain at right side of stilling basin headwall from spring located 247 feet left of dam Station 15+40 elevation 546 extend D.S. to discharge into paved ditch near headwall at Station 23+60.
Dike	Grouting Sand Gradation	27 Jun 75	ORLED	Due to consistent variation in materials, sand for grouting having 99 to 100% passing the No. 8 sieve and having a fineness modulas between 1.50 and 2.02, will be acceptable for grouting sand.
Dike	Zone 1 Grouting	8 Aug 75	ORLCD	Initiate 3 feet exploratory core drilling prior to drilling quadinary spaced grout holes zone one.
Dike	Modified Grouting Procedure Dike Left Abutment	1 Oct 75	ORLCD, ORLED	Trial grouting procedure added at the left abutment of dike to provide three 6-inch diameter grout holes through which a heavy sand mix is to be placed to seal the large openings.

TABLE XIII-1

RECORD OF DESIGN
(Continued)

AREA	DESCRIPTION OF CONTRACT WORK	DATE	ORGANIZATION UNIT REVIEW	COMMENTS
Dike	Stop Grouting	2 Dec 75	ORLED	Initiate stop grouting in 3-inch holes by zones Station 100+00 to 102+00 centerline only. Limit grout placed to 500 cubic feet per stage then wait until grout has set before pumping resumed. Due to excessive cost of Contractor's proposal to initiate this method, stop grouting was not initiated in this area. The 500 cubic feet limitation with normal contract method was continued.
Dike	Excavation Dike Inspection Trench	12 Mar 76	ORD, ORLED, ORLCD	It was agreed excavation of dike cutoff trench could proceed simultaneously with grouting of left abutment of dike, as restriction on Contract Drawing PR 18-12.5/21 only restricted excavation of inspection trench in the grouting area. Change back slopes in sandstone on right abutment to 1.5 H:1V.
Dike	Sand for Use with Mortar Grout	12 Apr 76	ORLED-G, ORLCD	Due to continuing sand gradation problems in connection with grout, 92% passing the No. 30 sieve is approved on an interim basis.
Dike	Cutoff Trench	18 May 76	ORLED, ORLCD	Enlarge cutoff trench from designed 15 feet width from Station *8+00 to 18+10 through the dike foundation* so that filter transition zones can be added along sides of cutoff trench.

TABLE XIII-1

RECORD OF DESIGN
EVALUATION AND CHANGES
DURING CONSTRUCTION

AREA	DESCRIPTION OF CONTRACT WORK	DATE	ORGANIZATION UNIT REVIEW	*COMMENTS*
Dike	Grouting Left Abutment of the Dike	24 Aug 76	OCE, ORD, ORLED, ORLCD	After much difficulty with the accomplishment of the grouting in this area, it was decided that only the upstream line will be completed in the second zone on 10-foot centers to bottom of Glen Dean limestone. If large voids or caverns are encountered, they will be filled with concrete. To further inhibit the possible seepage around left abutment, a 6-foot thick blanket of waste material would be placed along the upstream toe of the abutment from elevation 548 to 564. The blanket to extend 900 feet U.S. This blanket was not placed.
Dike	Exploratory Holes Left Abutment	5 Oct 76	ORLED, ORLCD	Four additional 6-inch diameter holes to be drilled in the left abutment of the dike to further inspection of grouting and facilitate use of focusing TV camera from WES.
Dike	Additional Piezometers Left Abutment of Dike	27 Oct 76	ORLED	Install additional piezometers. U.S. and D.S. of dike left abutment (Numbers WP-21 through WP-28).
Dike	Left Abutment Foundation Treatment	8 Mar 77	ORD, ORLED, ORLCD	It was considered that the grouted abutment was inadequate to protect against piping after impoundment. It was considered that the open cut method was that most positive assurance of adequate treatment.

TABLE XIII-1

RECORD OF DESIGN
(Continued)

AREA	DESCRIPTION OF CONTRACT WORK	DATE	ORGANIZATION UNIT REVIEW	COMMENTS
Dike	Positive Cutoff Left Abutment Dike	8 Mar 77	ORD, ORLED, ORLCD	Provide positive cutoff at left end of dike. Remove Glen Dean limestone Station 103+00$\pm$ to 102+30 and excavate to top of limestone to 100+40. Excavate side slopes on 1V to 1.5H to Station 102+30 and 1V to 1H from Station 102+30 to 100+40. Excavate end slope on 1V to 0.5H. Bottom of cutoff trench to be 24 feet wide. Dental concrete to be used to fill in low spots in remaining exposed top of limestone and seal all joints or cavities. Trench excavation to continue from present trench in right side of dike to provide construction access. Check will be made to determine if excavation will intercept the material in the downstream waste area which might cause excavation slope to be unstable. Later, the centerline was curved upstream to avoid excavating into the spoil area. District to use full flexibility to make appropriate changes as the conditions are further uncovered. Placement of waste fill on U.S. side of dike agreed to on 30 August is not necessary. Refill of trench excavation will consist of normal dike section with central impervious core protected on D.S. side by a two zone filter.

TABLE XIII-1

RECORD OF DESIGN
(Continued)

AREA	DESCRIPTION OF CONTRACT WORK	DATE	ORGANIZATION UNIT REVIEW	COMMENTS
Dike	Right Abutment Treatment	16 May 77	ORD, ORLED, ORLCD	After excavating for design grade and further air track exploration, it was decided to excavate vertically in the Mansfield sandstone to top of the Glen Dean limestone at about Station 18+56. The cleaned out solution channel would be backfilled with concrete and a massive concrete fillet to be placed on top of the limestone elevation 531 against the cut in sandstone. Place Indiana No. 4 stone in the U.S. cavities and place dental concrete in downstream cavities of excavated cutoff trench from 8+00 to 18+10±. Clean collapsed areas U.S. and D.S. and use reverse filters as needed. If openings are large, minus 8 material from riprap bedding may be required under Indiana No. 4 stone. Place concrete bulkhead 4 feet thick with reinforcement of 1 inch bars on 12-inch centers in both directions overend of cutoff trench at Station 18+00 from elevation 506 to 531. Bulkhead will be wrapped around U.S. wall to cover three open solution joints. A second placement using 1-1/8 inch reinforcing mat will fill entire notched sump area ahead of 18+00.
Dike	Left Abutment Excavation	19 May 77	ORD, ORLED	Construction Division is to perepare a detailed report covering the left abutment grouting operations on distribution and effectiveness of the grout as found during the excavation.

TABLE XIII-1

RECORD OF DESIGN
(Continued)

AREA	DESCRIPTION OF CONTRACT WORK	DATE	ORGANIZATION UNIT REVIEW	COMMENTS
Dike	Left Abutment Excavation	21 Jun 77	ORD, ORLED, ORLCD	After explorations, it was decided to remove Glen Dean limestone from 103+00± to 100+80 instead of just back to 102+30 as decided on 28 March 1977.
Dike	Buried Valley Excavation	17 Jun 77	ORD, ORLED, ORLCD	Discussion of removing saturated sand above Hardensburg Shale between Station 4+00 to 7+50. Study conditions between 4+00 and 7+50 to determine if sand can be stabilized so that the excavation can be stopped.
Dike	Treatment of Right Abutment	6 Jul 77	ORD, ORLED, ORLCD	Grout pipes will be installed through the solution joint prior to placement of concrete so that the bottom 4 feet of the limestone can be grouted below the concrete.
Dike to Dam	Grouting	2 Dec 75	ORLED	Initiate stop grouting in 3-inch holes by zones on "A" and "C" lines Station 117+45 to 144+00. Limit grout placed to 500 cubic feet per stage, then wait until grout has set before resuming pumping. Due to excessive cost of contractor's proposal to initiate this method, stop grouting was not initiated in these areas.
Dike to Dam	Grouting	5 Apr 76	ORLCD	In lieu of using packers at the top of the curtain as called for in the contract, a 1-1/2-inch casing will be set from the ground surface to the top of the grout curtain and all grouting will be performed in stages with the grout introduced at the top of the casing. Letter of understanding.

TABLE XIII-1

RECORD OF DESIGN
(Continued)

AREA	DESCRIPTION OF CONTRACT WORK	DATE	ORGANIZATION UNIT REVIEW	COMMENTS
Dike to Dam	Grouting	30 Aug 76	ORD, OCE, ORLED, ORLCD	It was decided to reduce the 3-line grout curtain to one line (upstream line) of vertical holes on 10-foot centers. Where large caves or cavities are encountered, the hole would be marked to be filled with concrete.
Dike to Dam	Grouting	31 Aug 76	ORLED	Neat cement grout placement to be limited to 200 cubic feet per stage and sanded grout to be limited to 500 cubic feet per stage unless hole is tightening up in which case limits can be increased 25%.
Dike to Dam	Installation of Weir	18 Nov 76	ORLED	Install a weir at the entrance to the Robert Hall Spring in order to accumulate sufficient records on flow prior to impoundment.
Dike to Dam	Grouting	8 Mar 77	ORD, ORLED, ORLCD	Present grouting methods were considered to be completely inadequate and further pursuit of stage grouting with thin grout was considered to be of no value. Information is needed on water levels in various formations to establish the need for and type of grouting program needed. It was agreed that the cavernous condition near the left side of the spillway had to be corrected prior to impoundment above elevation 514. Grouting by the 3-line stage grouting method necessary at the right abutment of the dike.
Dike to Dam	Exploration	8 Mar 77	ORD, ORLED, ORLCD	An exploration program will be started immediately to better define the bedrock and groundwater conditions between the dam and dike. Piezometers will be set between the dike and dam in the various formations to determine the preimpoundment water levels in the Mansfield sandstone, Glen Dean limestone and Golconda limestone. These piezometers to be set U.S. and D.S. of the grout line so they will not be grouted up and provide useful information on changes in the groundwater levels from the impoundment. (Piezometers No. 40 through 50).

TABLE XIII-1

RECORD OF DESIGN
(Continued)

AREA	DESCRIPTION OF CONTRACT WORK	DATE	ORGANIZATION UNIT REVIEW	COMMENTS
Dike to Dam	Foundation Treatment	6 Jul 77	OCE, ORD, ORLED ORLCD, WES	DIKE TO DAM TREATMENT METHOD POSSIBILITIES (1) Grouting (2) Concrete Wall Through the Glen Dean Limestone (3) Raise Pool to Elevation 536 to Determine Leakage Loss
		6 Jul 77	ORLED, ORLCD	Drill air track exploratory holes below the 560 contour from the dike right abutment to the spillway to gain additional information for determining alignment for proposed cutoff wall. Decision to start test trench in spillway.
		3 Oct 77	ORLED, ORD (27 Sep 77)	Addition of a positive concrete cutoff wall between the right abutment of the dike and the spillway control structure. The wall is to extend from bottom of Glen Dean limestone to elevation 536 or top of limestone, whichever is higher. Impervious material is to be added above wall to provide protection to spillway control grade elevation 548.
Spillway	Borrow	3 Dec 75	ORLED	An additional borrow is provided which will be obtained by widening the spillway 50 feet on right side from Station 0+00 to Station 6+00 as located on Contract Drawing PR 18-12.5/2. The approval is based on no additional cost to the Government.

TABLE XIII-1

RECORD OF DESIGN
(Continued)

AREA	DESCRIPTION OF CONTRACT WORK	DATE	ORGANIZATION UNIT REVIEW	COMMENTS
Spillway	Borrow	6 May 76	ORLED	Widening of the right side of the spillway between Stations 6+00 and 22+50. Widening of the spillway will provide the additional rock necessary to complete the embankment due to the changes in the treatment of the dam foundation of 20 April 1976.
Spillway	Exploration	19 May 77	ORD, ORLED, ORLCD	Explore by air track drilling the condition of the foundation of the designed sill. Decide if any change is needed in depth of sill.
Spillway	Control Sill Location	7 Sep 77	ORD, ORLED, ORLCD	It was decided to move the control sill to Station 7+50 of the spillway.
Spillway	Spillway Sill	14 Sep 77	ORLED	The crest structure will be founded on a test section of cutoff wall extending through the Glen Dean limestone where constructed 50 feet from the left side of the spillway to 15 feet from the right side of the spillway and on limestone for the remainder of the spillway width. The structure will also extend 10 feet into each side slope of the spillway.
	DESCRIPTION		CONSULTANTS	
All Areas of Foundation Treatment	Review of Foundation Treatments	30 Jul 77	(1) K.S. Lane Consulting Engineer (2) J.M. Kellberg Consulting Geologist	See consultant's report in Appendix.
Spillway to Dam Station 141+00	Proposed Grouting	22 Jun 78	ORLED-G, ORDED	Based on data furnished by ORLED, the proposed completion of grouting should not be performed at this time.

TABLE 13-2

CONTRACT MODIFICATIONS AFFECTING THE FOUNDATION

1. Modification #P00001, Case 101. This modification provided for the installation of a wellpoint piezometer in the center of the Stage I cofferdam area to monitor the effect of the dewatering system during excavation. This work was accomplished between 31 May and 3 June 1975.

2. Modification #P00004, Case 104. This modification provided for the construction of a 6-8 ft. inspection trench along the centerline of the Stage II cofferdam. The purpose of this inspection trench was to remove the farm drainage tile from beneath the cofferdam. This work was performed as part of the Stage II cofferdam foundation stripping.

3. Modification #P00007, Case 2. An additional borrow area, designated as borrow area 3A, was made available as a source for impervious material for completion of the Stage II cofferdam and as a source of rock for use by the contractor in haul road construction.

4. Modification #P00009, initial and final, Case 4. This modification revised the impervious core of the dam from 1 on 10 slopes to vertical. As a result, the rock excavation limits were changed slightly. Also, Casagrande piezometer locations were changed.

5. Modification #P00011, Case 107. Modified grouting procedure to drill 3 each 6-inch holes on the left abutment of the dike grout line and to backfill these holes with ready-mix grout. This work was accomplished between 19 November and 29 December 1975.

6. Modification #P00012, Case 109. Modified contract to perform dam left abutment exploratory work. Needed to outline the extent of weathering along joints on the left abutment of the dam. This work

was performed by cleaning between the jointed rock prior to presplitting. Additional air-trac holes were also drilled in this area. This work was accomplished between 3 March and 10 March 1976.

7. Modification #P00013, Case 14. Provided for widening of the spillway 50 feet between station 0+00 and 6+0. Also, certain other areas, sta. 6+0 to 22+50, were made available for widening (50 ft.) to provide a source of rock to replace rock utilized in the contractor's haul roads. The extra rock produced was used by the contractor for haul road construction.

8. Modification #P00014, Case 8. This modified the contract to provide for the major portion of the revised dam foundation treatment. Work included special treatment of the Beech Creek Limestone-Big Clifty Sandstone contact around the perimeter of the lower dam excavation. Cleaning, dental concrete, and reverse filters were part of the actual work performed. This work was accomplished concurrent with the Mod. #P00015 work described below.

9. Modification #P00015, Case 6. Work covered by this modification included extending and widening the cutoff trench, extending cut into the lower left abutment, and placing concrete wall slabs on the Beech Creek Limestone at both ends of the trench. Also included in this modification were the changes to the presplit slopes and the extended excavation limits in the upper two limestones on the left abutment of the dam. The cutoff trench portion of this and the preceeding modification was accomplished primarily between May and July 1976. The presplitting and excavation of the upper limestones was accomplished later in the 1976 construction season.

10. Modification #P00016, Case 9. The grouting of the dam foundation was modified along with the foundation changes in the preceeding two modifications. Basically, the curtain was deepened to extend

through the Beech Creek Limestone further into both abutments. Work on this change was accomplished during the 1976 construction season.

11. Modification #P00017, Case 110. This modification provided payment for four additional piezometers installed upstream of the dam, near the toe of the Stage I cofferdam. These piezometers were intended to supplement the contractor's excavation piezometers installed as part of the dam overburden excavation. All of these piezometers were temporary in nature. Work on this modification was performed between 16 February and 26 February 1976.

12. Modification #P00018, Case 111. Work under this modification included filling of the cavities on the upstream and downstream walls of the dam cutoff trench with lean concrete. This concrete was placed by means of simplified construction expedient forms. Work on this modification was basically performed between 17 June and 20 July 1976.

13. Modification #P00019, Case 12. The dike foundation on the right abutment was modified to provide for reverse filters against the downstream limestream face. To accomplish this, the cutoff trench was widened and the excavation side slopes were changed. Work on this modification was performed during the 1976 and 1977 construction seasons.

14. Modification #P00020, Case 13. This modification provided payment for extending the cut in the Golconda Limestone another 30 feet landward of the Mod. #P00014 excavation limits. This modification was completed during the 1976 construction season.

15. Modification #P00021, Case 15. This modification was the result of a Value Engineering change proposed by the contractor. The change consisted of steepening the dam overburden excavation slopes from a 3:1 to a 1:1 slope in the lower portion of the excavation. Work on this modification coincided with overburden excavation.

16. Modification #P00024, Case 17 and Modification #P00035, Case 120. Upon termination of the grouting program at the left abutment of the dike, further foundation information was needed to evaluate future design changes. As part of this foundation evaluation, four each additional 6-inch core holes were drilled on the left abutment of the dike. This modification authorized payment for these core holes. Work on this modification was performed between 22 November 1976 and 3 March 1977.

17. Modification #P00025, Case 115A and Modification #P00045 Case 124. The dam contract was modified to install a weir at the downstream cave entrance at Hall Spring. This weir was intended to monitor the fluctuations in the flow of this spring. Work on this weir was performed between 6 and 16 December 1976. Later this weir was raised 1 foot in height when it was realized that the weir was not large enough to measure the spring flow during periods of heavy rainfall.

18. Modification #P00030, Case 22. This modification included miscellaneous revisions to the dam and dike. On the dam, the downstream slope, movement and reference monuments were revised. On the dike, the left abutment was excavated to shale, the overburden between stations 4+0 and 8+0 was removed to rock, and dental treatment with reverse filters were performed in the excavated limestone. Work on the dike portion of this modification was started on March 29, 1977 and continued through most of the 1977 construction season. Monument revisions were accomplished during final phases of dam construction.

19. Modification #P00032, Case 112. During grouting operations the necessity to modify the contract to allow circuit grouting was realized. Circuit grouting was performed through loose casings and mud seams. This work was performed throughout the grouting phase of this contract.

20. Modification #P00033, Case 116. Piezometers were modified and a wellpoint installed and an NX core hole was added to the left abutment of the dam. Work on this modification was performed during the 1977 and 1978 construction season.

21. Modification #P00036, Case 119. As part of the preliminary work for the cutoff trench, a series of air-trac exploratory holes were drilled between the dike and the spillway. This modification provided funds to accomplish this work. The exploratory holes were drilled between 3 August and 12 September 1977.

22. Modification #P00037, Case 23. This modification changed the dam and spillway contract to install a series of 21 piezometers in the area of the dike and the spillway. These piezometers were installed adjacent to the uncompleted groutline to determine rock and groundwater conditions in the area for further treatment revisions. This work was performed between 12 May and 2 June 1977.

23. Modification #P00039, Case 27. This modification changed the contract to include a positive cutoff from the dike to the spillway. The work on this cutoff trench was performed between October 1977 and September 1978.

24. Modification #P00040, Case 121. This modification changed the location of the Type B accelerograph shelter to the top of the left abutment at elevation 561. Work on this accelerograph shelter was performed during the 1978 construction season.

25. Modification #P00041, Case 122. This modification deleted the short cutoff wall at the left abutment of the dam at elevation 564. The Type B accelerograph shelter in the above modification replaced the cutoff wall.

26. Modification #P00042, Case 24. This modification included foundation changes on both the dam and the dike. At the right

abutment of the dike, the collapse zone was cleaned out and a reinforced concrete wall was placed over the limestone and sandstone collapse structure. Following the treatment at the right abutment, the limestone and sandstone were grouted through pre-installed grout pipes. On the dam left abutment, the concrete wall over the Glen Dean Limestone was shortened, overhangs and cavities beyond the wall were treated only as necessary. Work on this modification was performed during the 1977 construction season.

27. Modification #P00043, Case 26. As part of the design phase of the cutoff trench, a test trench across the spillway at station 7+50 was excavated and filled with concrete. This modification enabled this trench to be included in the dam and spillway contract. In addition, the spillway control sill was moved from station 10+70 to the top of the test trench. Work on this modification was started 15 August 1977 and was completed by 15 June 1978.

28. Modification #P00044, Case 123. This modification provided a 12-inch drain for the spring encountered in the Golconda Limestone near the toe of the right abutment of the dam. This spring was routed to empty into the paved ditch along the right abutment. Work on this drain was started on 22 May 1978 and the installation of this pipe was completed on 26 May 1978.

29. Modification #P00049, Case 30. This modification provided for payment of the overruns in limestone excavation at the right abutment of the dike (station 8+0 to 18+00) that occurred during Modification #P00019 excavation.

30. Modification #P00056, Case 32. This modification provided for payment of cumulative changes made in procedures during the drilling and grouting system.

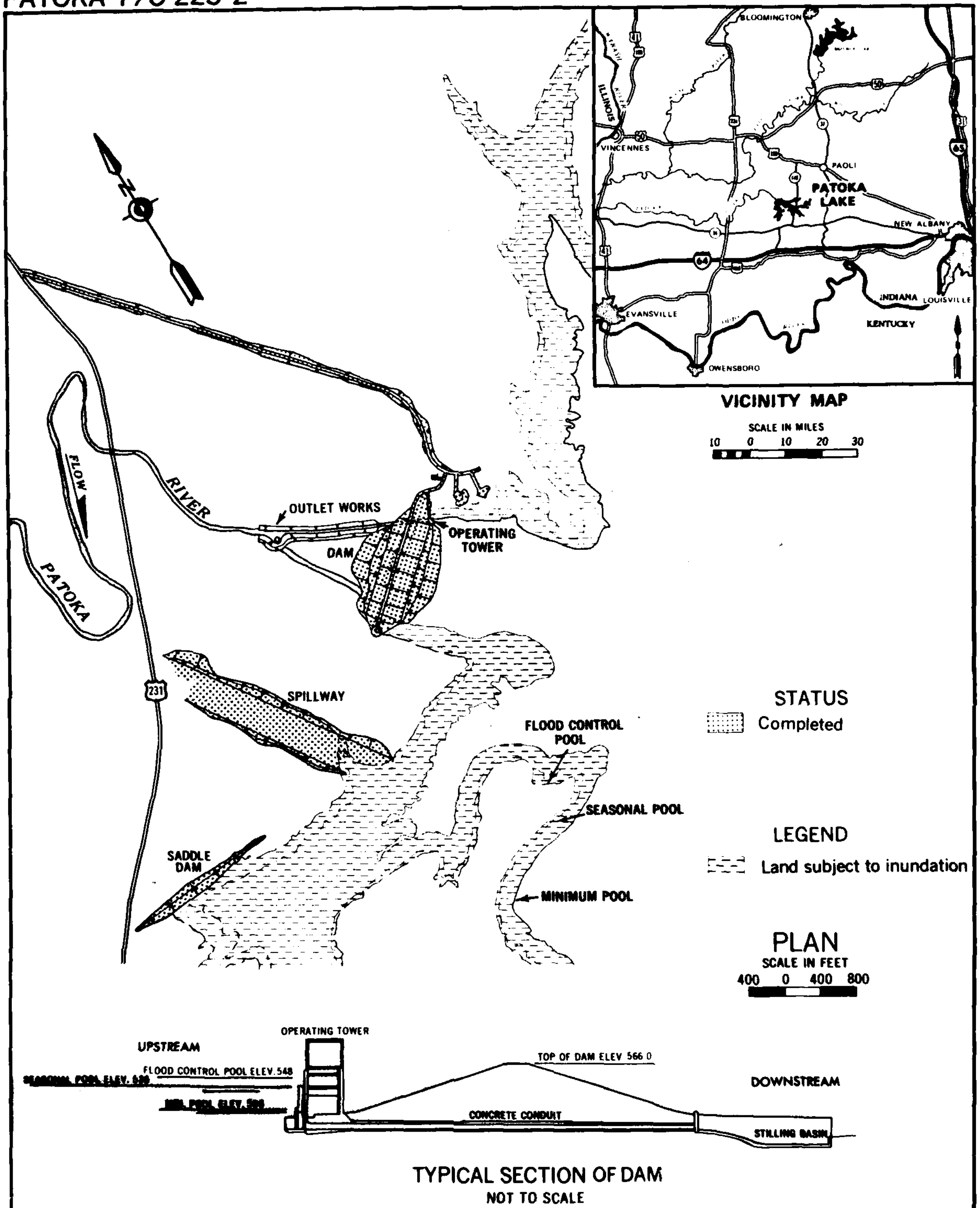
APPENDIX A

GENERAL INFORMATION TO PATOKA
LAKE FOUNDATION REPORT

Table of Contents

C10847

AERIAL VIEW OF PROJECT



PATOKA RIVER, IND.

PATOKA LAKE

LOUISVILLE, KY. DISTRICT

SCALE AS SHOWN

REVISED 30 SEPTEMBER 1979

PATOKA LAKE, INDIANA

Condition of Improvement, 30 September 1979

LOCATION: The damsite is 3/4 mile northeast of Ellsworth, Indiana, 118.3 miles above the mouth of Patoka River, a tributary of the Wabash River.

AUTHORIZATION: The project was authorized by the Flood Control Act approved 27 October 1965 (Public Law 89 298, 89th Congress).

PROJECT FEATURES: The lake will serve the purposes of flood control, general recreation, fish and wildlife recreation, water supply and water quality control. Operation of the project for flood control will control the maximum flood of record at the damsite and will reduce the stage of a flood equal in magnitude to the 1961 flood by 3.3 feet at Jasper and 1.0 foot at Winslow.

MULTIPLE-PURPOSE PROJECT:
Counties affected: Dubois, Orange and Crawford Counties, Indiana.

Operating Levels

Pool	Elevation of Pool	Capacity	Area	Backwater Main Stream
		(acre-feet)	(acres)	(length-mile)
Minimum	506	13,250	2,010	11
Water Supply	506-536	167,290	8,880	25
Flood Control	536-548	121,100	11,300	31
Total Storage	548	301,640	11,300	31

Drainage area above dam – 168 square miles.

Dam: Earth and rock fill 1,550 feet in length and a maximum height of 84 feet.

Spillway: Located through the left abutment with a width of 350 feet, length of 2,800 feet, and a maximum depth of cut 90 feet.

Outlet Works: Control tower with two service gates 4' x 12' discharging to an 8' x 12' oblong concrete conduit along the right abutment leading to the stilling basin.

Dike: Earth fill 1,600 feet in length and a maximum height of 566 feet.

Relocations: Relocation or alteration is necessary for 6.6 miles of state highway, and 1.6 miles of county roads. There are about 16 miles of power lines and about 14 miles of telephone lines that will require some alterations. Four cemeteries were moved.

Land (Fee and Lesser Interests):

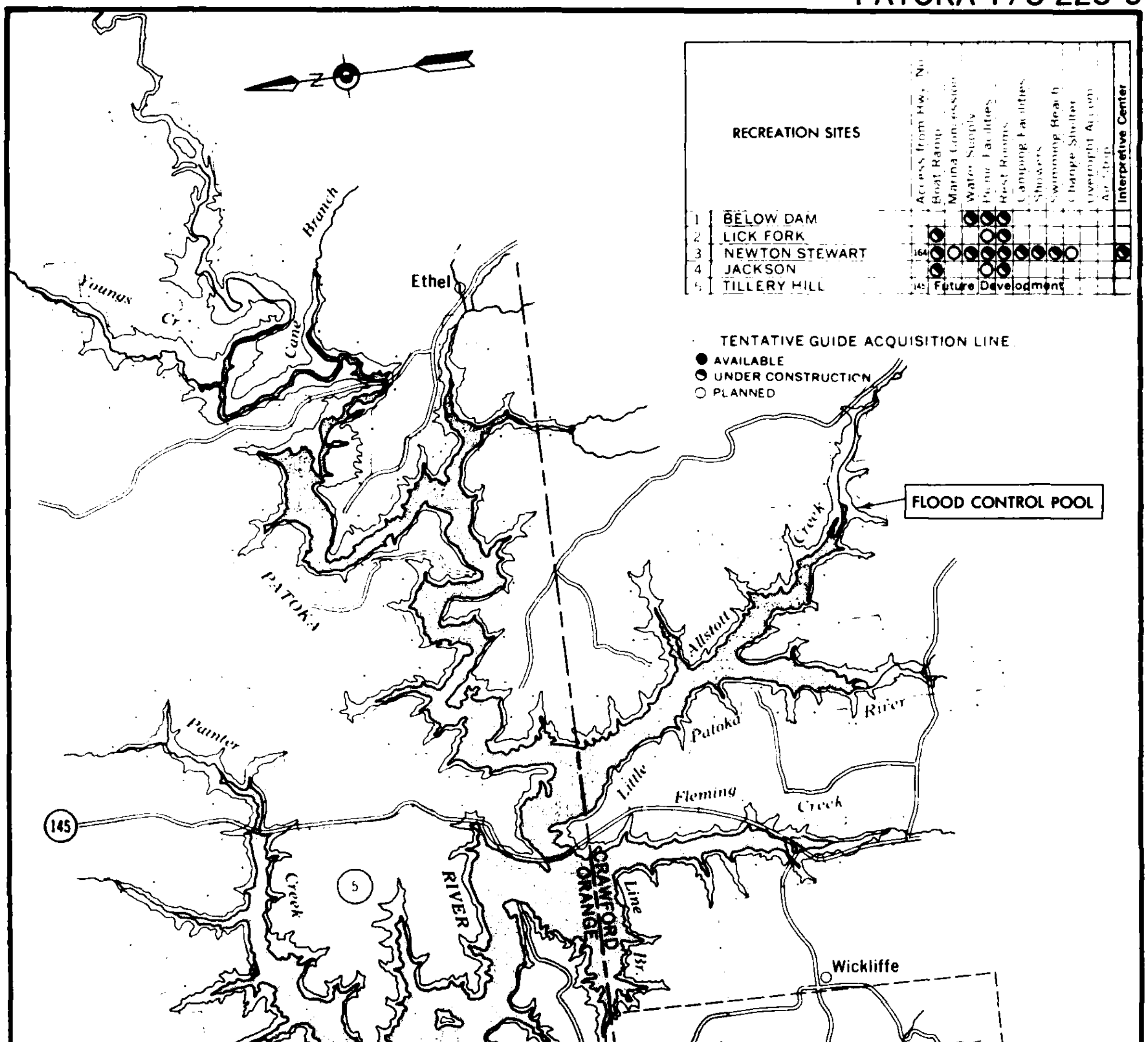
	Acquisition to Date	Estimated for Total Project
Fee	26,470.26 Acres	26,835.46 Acres
Flowage easement	194.02 Acres	229.89 Acres
Other easement	78.75 Acres	149.03 Acres

PATOKA F/C 223-4

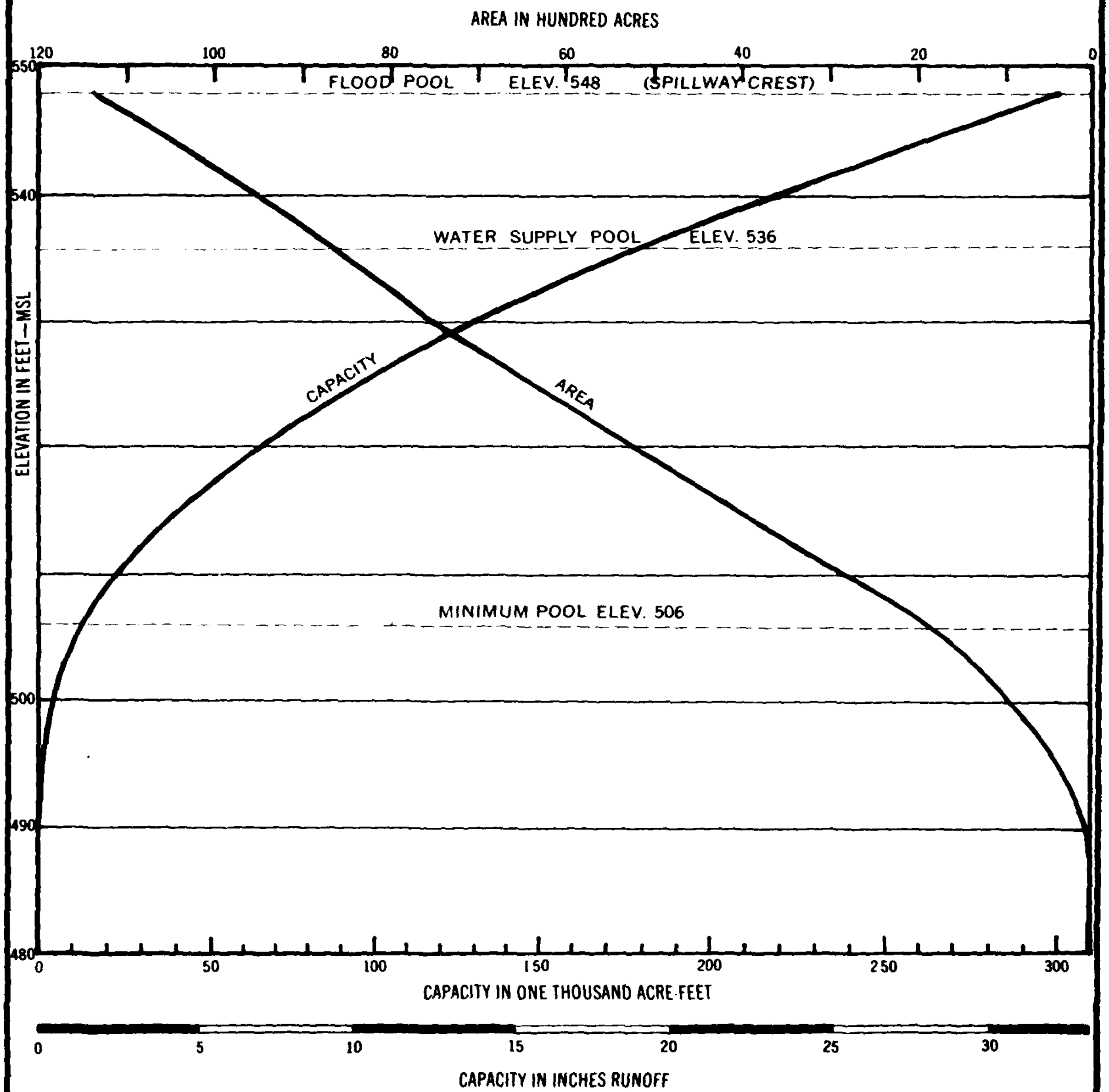
PROGRESS: Construction of outlet works, dam, saddle dam, and spillway are complete. Relocation of roads, utilities, and recreation facilities are under construction. Lake impoundment, January 1978.

COST DATA:

Estimated Federal Cost (1979)	$54,500,000
Estimated Nonfederal Cost (1979)	24,401,000
Estimated Project Cost (1979)	79,906,000
Federal Costs to 30 September 1979	45,141,508
Federal Net Allotments to 30 September 1979	48,505,920


RECREATION SITES
Access from Hwy. No.
Boat Ramp
Marina Concession
Water Supply
Picnic Facilities
Rest Rooms
Camping Facilities
Showers
Swimming Beach
Change Shelter
Overnight Accom.
Air Strip
Interpretive Center
1 BELOW DAM
2 LICK FORK
3 NEWTON STEWART 164
4 JACKSON
5 TILLERY HILL 145 Future Development
TENTATIVE GUIDE ACQUISITION LINE
AVAILABLE
UNDER CONSTRUCTION
PLANNED
N
Youngs Cr.
Branch
Cane
Ethel
PATOKA
Allstott
Creek
FLOOD CONTROL POOL
Little Patoka River
Panter
Fleming Creek
145
Creek
RIVER
CRAWFORD
ORANGE
CO.
CO.
Line Br.
Wickliffe
164
Creek
Hoosier National Forest
Ritter
SEASONAL POOL
4
3
5
Cr.
Fork
DUBOIS CO.
Lick
DAM
1
2
Hoosier National Forest
SPILLWAY
DIKE
SCALE IN MILES
1 0 1
PATOKA RIVER, IND.
PATOKA LAKE
LAKE AREA
LOUISVILLE, KY. DISTRICT
SCALE AS SHOWN
REVISED 30 SEPTEMBER 1979

PATOKA F/C 223-6



PATOKA RIVER, IND.

PATOKA LAKE

POOL DATA

LOUISVILLE, KY. DISTRICT

SCALES AS SHOWN

REVISED 30 SEPTEMBER 1979

OPERATING RELEASES

DESIGN MAXIMUM RATE:

Dec.-Mar. 1200 C.F.S.
Apr.-Nov. 850 C.F.S.

DESIGN MINIMUM RATE:

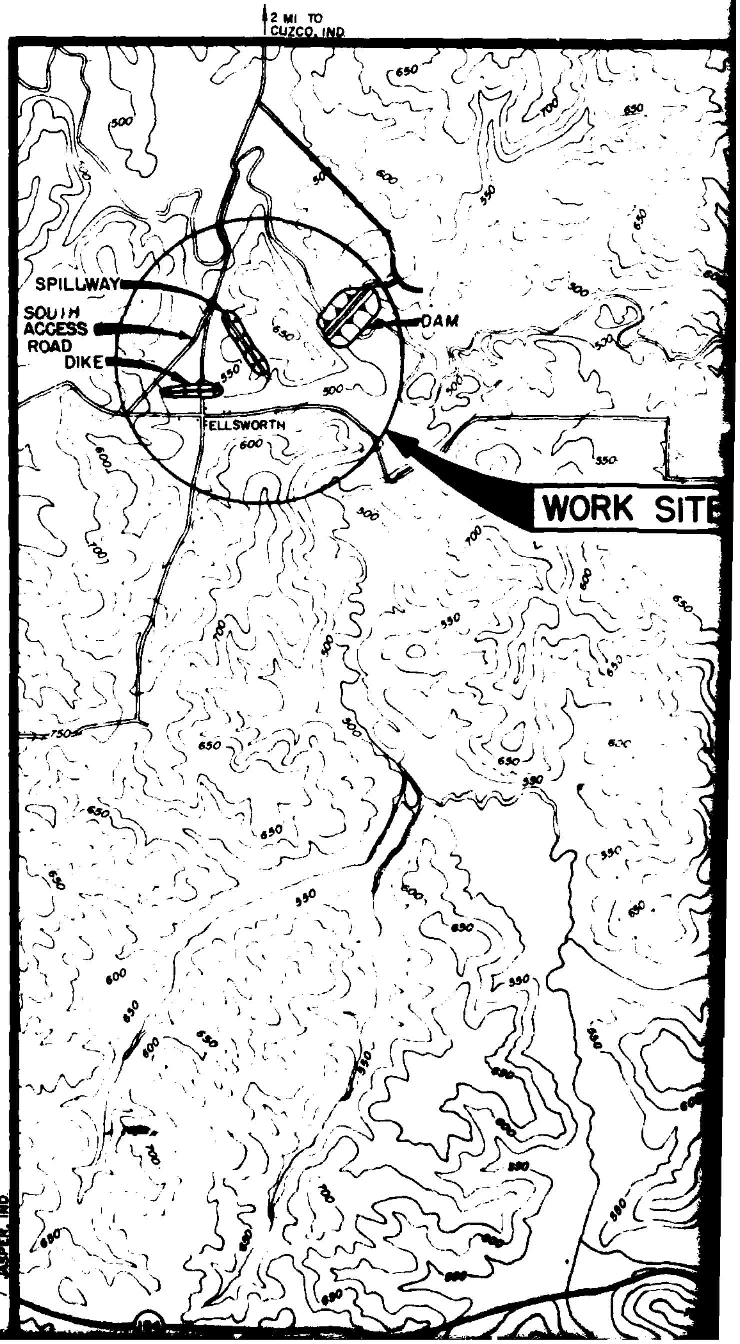
5 C.F.S.

CORPS OF ENGINEERS

PATO
PATOKA
DAM

INDEX TO DRAWINGS

SHEET NO.	DESCRIPTION
18-12.5/1	LOCATION PLAN, VICINITY MAP & INDEX
2	GENERAL PLAN
3	GRADING PLAN FOR WASTE AREAS
4	BORROW AREA, BORING LOCATION PLAN, HAUL ROAD & WORKING LIMITS
5	CONSTRUCTION SEQUENCE STAGE I
6	CONSTRUCTION SEQUENCE STAGE II
7	CONSTRUCTION SEQUENCE STAGE III
8	CONSTRUCTION SEQUENCE STAGE IV
9-14	HYDROGRAPHS
15	PLAN OF DAM
16	PLAN OF DIKE AND SERVICE ROAD TO DIKE
17	SPILLWAY PLAN & PROFILE AND DETAILS
18	TOE DRAINAGE SYSTEM
19	EXCAVATION PLAN OF DIKE
20	DAM-ABUTMENT TREATMENT SECTIONS
△3 21	GROUTING PLAN AND PROFILE
22	TYPICAL DAM SECTIONS
△5 23	TYPICAL DIKE SECTIONS
24	TYPICAL SPILLWAY SECTIONS
25	OUTLET CHANNEL PLAN
26	OUTLET CHANNEL PLAN
27	OUTLET CHANNEL PLAN
28	INLET & OUTLET CHANNEL SECTIONS
29	OUTLET CHANNEL SECTIONS
30	OUTLET CHANNEL SECTIONS
31	OUTLET CHANNEL STAIRWAY DETAILS
32	BELOW DAM ACCESS ROAD PLAN & PROFILE
33	BELOW DAM ROAD SECTIONS
34	MISCELLANEOUS DETAILS
35	ACCELEROGRAPH SHELTERS
△6 36	SPILLWAY CONTROL SILL AND SPILLWAY SERVICE ROAD PLAN & DETAILS
37	INSTRUMENTATION PLAN OF DAM AND DIKE
38	INSTRUMENTATION SECTIONS OF DAM & DIKE
39	INSTRUMENTATION DETAILS
△1 40	TRASH BOOM
41	GUARD POST RACKS DETAILS & INSTALLATION
42	SOUTH ACCESS ROAD PLAN & PROFILE
43	SOUTH ACCESS ROAD PLAN & PROFILE
44	SOUTH ACCESS ROAD SECTIONS
45	SOUTH ACCESS ROAD SECTIONS
△4 46	SOUTH ACCESS ROAD SECTIONS
47	BORING LOCATION PLAN – DAM
48	BORING LOCATION PLAN – DIKE
49	BORING LOCATION PLAN – SPILLWAY
50	SOIL CLASSIFICATION CHART AND LEGENDS
51-92	BORING LOGS
△1 100	GASOLINE DISPENSING UNIT & SECTIONS
	STATE STANDARD DRAWINGS
93-98	MISCELLANEOUS STANDARDS SHEET MB, GR-1, GR-10, ME-2, ME-1, MN
	REFERENCE DRAWINGS
PR 18-12.1/10	OUTLET WORKS EXCAVATION PLAN & PROFILE BORING LOCATION
14	STILLING BASIN PLAN & PROFILE
15	STILLING BASIN MASONRY DETAILS



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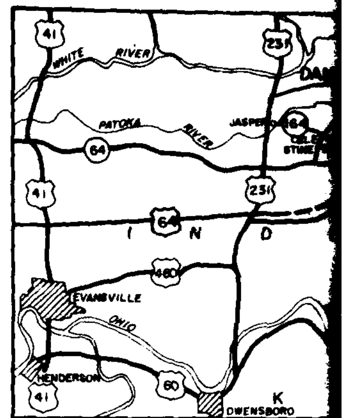
M AND SPILLWAY



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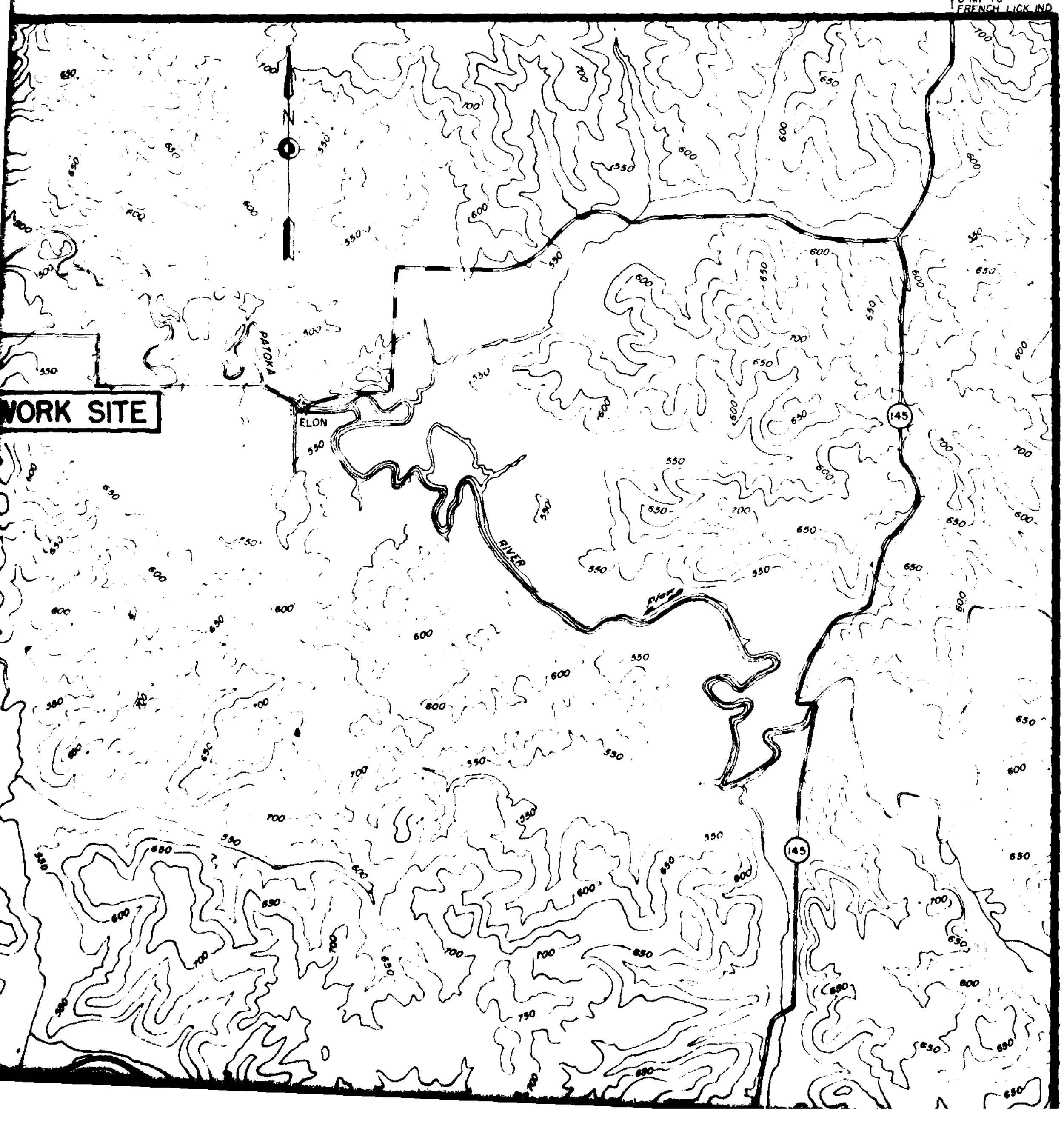

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	19SEP77	SHEET
	22MAR78	SHEET

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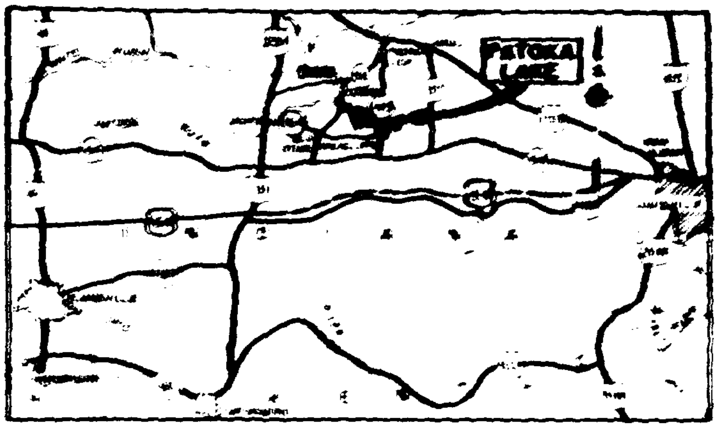

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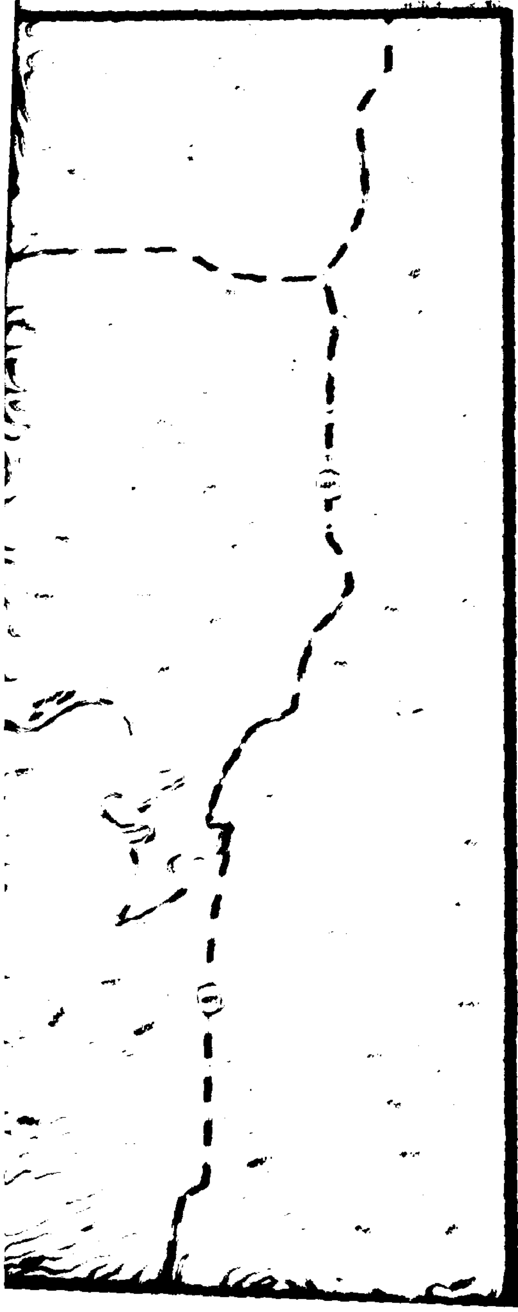
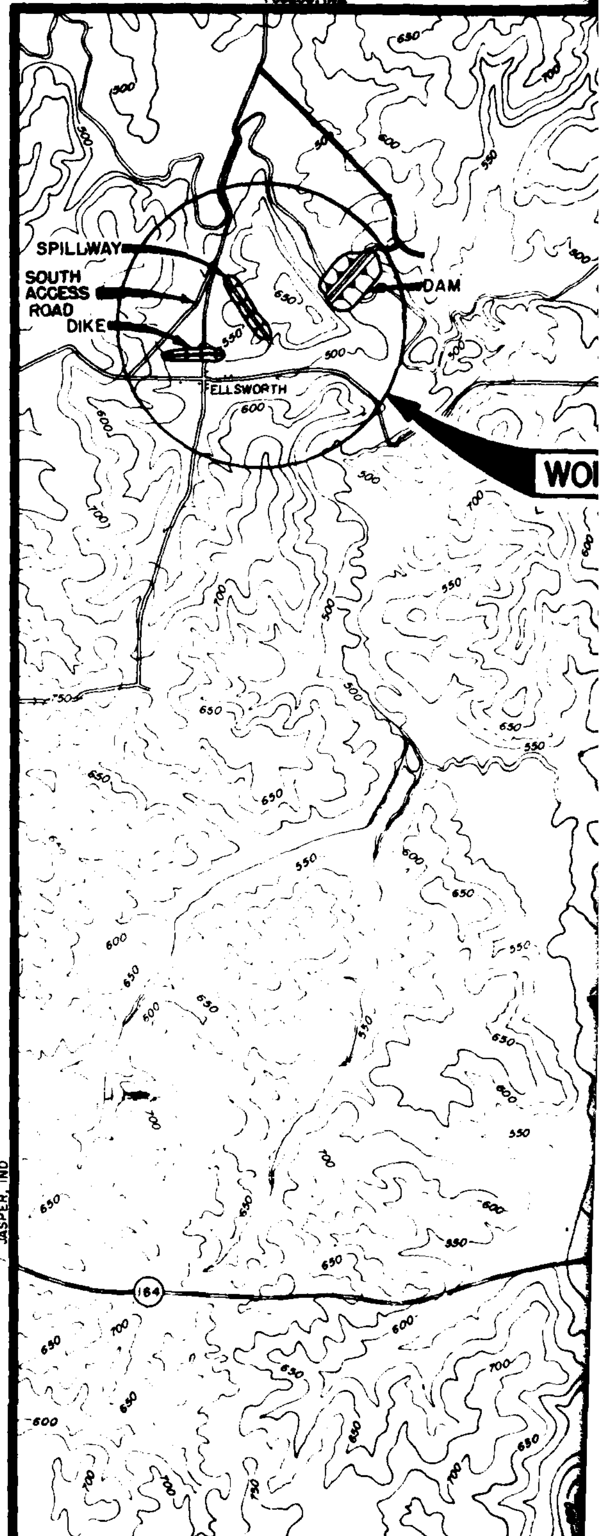


19	EXCAVATION PLAN OF DIKE
20	DAM-ABUTMENT TREATMENT SECTIONS
21	GROUTING PLAN AND PROFILE
22	TYPICAL DAM SECTIONS
23	TYPICAL DIKE SECTIONS
24	TYPICAL SPILLWAY SECTIONS
25	OUTLET CHANNEL PLAN
26	OUTLET CHANNEL PLAN
27	OUTLET CHANNEL PLAN
28	INLET & OUTLET CHANNEL SECTIONS
29	OUTLET CHANNEL SECTIONS
30	OUTLET CHANNEL SECTIONS
31	OUTLET CHANNEL STAIRWAY DETAILS
32	BELOW DAM ACCESS ROAD PLAN & PROFILE
33	BELOW DAM ROAD SECTIONS
34	MISCELLANEOUS DETAILS
35	ACCELEROGRAPH SHELTERS
36	SPILLWAY CONTROL SILL AND SPILLWAY
	SERVICE ROAD PLAN & DETAILS
37	INSTRUMENTATION PLAN OF DAM AND DIKE
38	INSTRUMENTATION SECTIONS OF DAM & DIKE
39	INSTRUMENTATION DETAILS
40	TRASH BOOM
41	GUARD POST RACKS DETAILS & INSTALLATION
42	SOUTH ACCESS ROAD PLAN & PROFILE
43	SOUTH ACCESS ROAD PLAN & PROFILE
44	SOUTH ACCESS ROAD SECTIONS
45	SOUTH ACCESS ROAD SECTIONS
46	SOUTH ACCESS ROAD SECTIONS
47	BORING LOCATION PLAN – DAM
48	BORING LOCATION PLAN – DIKE
49	BORING LOCATION PLAN – SPILLWAY
50	SOIL CLASSIFICATION CHART AND LEGENDS
51-92	BORING LOGS
100	GASOLINE DISPENSING UNIT & SECTIONS
	STATE STANDARD DRAWINGS
93–98	MISCELLANEOUS STANDARDS SHEET MB, GR-1,
	GR-10, ME-2, ME-1, MN
	REFERENCE DRAWINGS
PR 18-12.1/10	OUTLET WORKS EXCAVATION PLAN & PROFILE
	BORING LOCATION
14	STILLING BASIN PLAN & PROFILE
15	STILLING BASIN MASONRY DETAILS
17A	STILLING BASIN WINGWALLS
19	OPERATING TOWER-MASONRY VERTICAL SECTION
	A-A & HORIZ. SECTION AT EL. 520.0
20	OPERATING TOWER-MASONRY TRANSVERSE VERT.
	SECTIONS B-B, C-C & C'-C'
21	OPERATING TOWER-MASONRY TRANSVERSE VERT.
	SECTIONS D-D & E-E
22	OPERATING TOWER-MASONRY HORIZONTAL SECTION
	AT EL. 487.0
11-12	CONDUIT CROSS SECTIONS
36A	TYPICAL SECTIONS - SPILLWAY
54A	BORROW AREA AND MISCELLANEOUS BORING LOGS
PR 18-12.5/21 A	GROUTING PROFILE - LEFT ABUTMENT & VALLEY
PR 18-12.5/46A	BIG SPRINGS WEIR - PLANS, PROFILE & SECTION
PR 18-12.5/23A	TYPICAL DIKE SECTIONS



INDEX TO DRAWINGS

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PR 18-12.5/103-104	PLAN
105-109	SECTIONS

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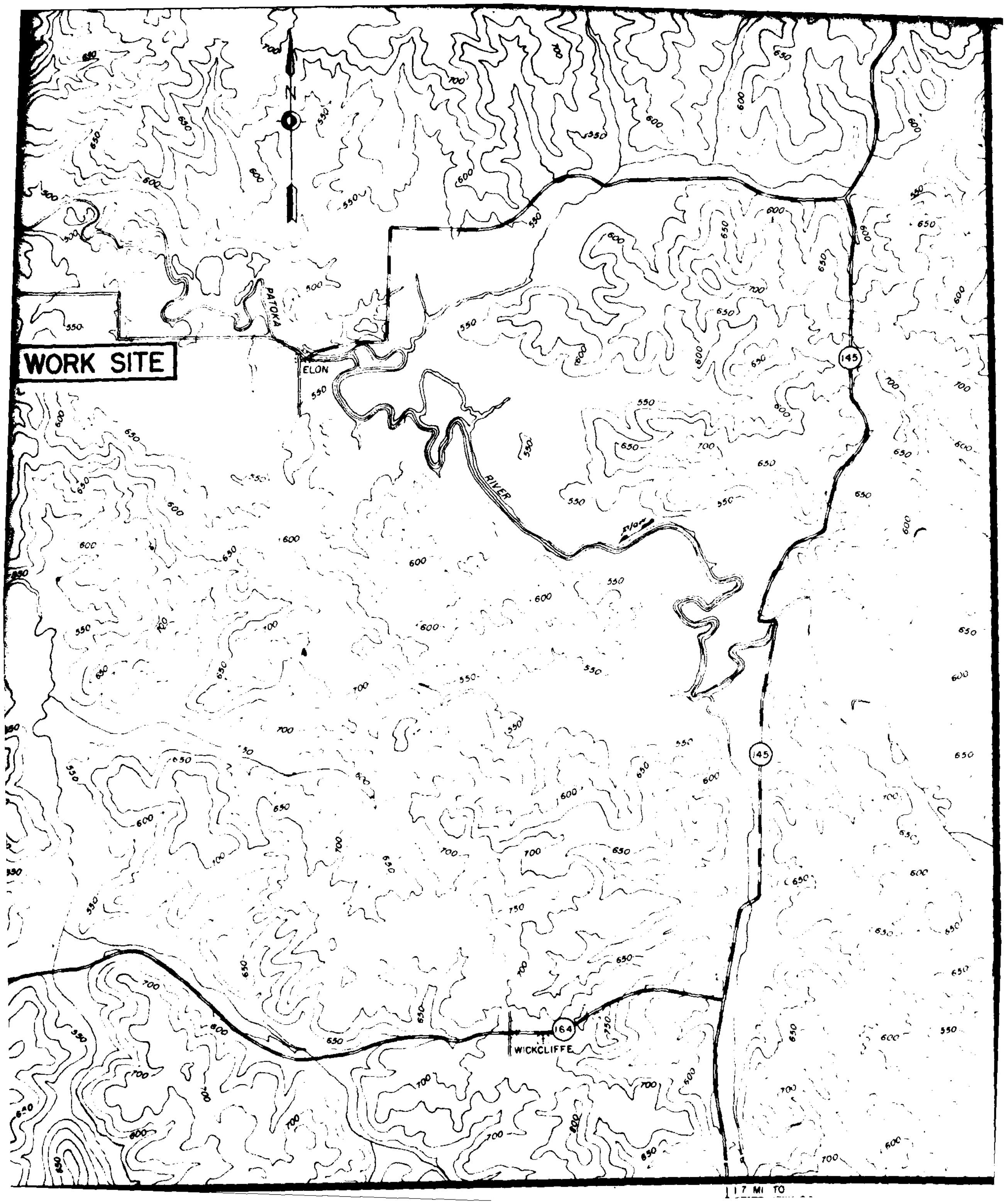

N
WORK SITE
PATOKA
ELON
RIVER
145
164
WICKCLIFFE
17 MI TO

THE APPROVAL SIGNATURES APPEARING IN THE TITLE BLOCK OF THIS SHEET CONSTITUTE APPROVAL OF ALL DRAWINGS LISTED IN THE INDEX.

REVISION	DATE	DESCRIPTION	BY
	[illegible]	SHEET ADDED (CONT MOD)	E.M
	[illegible]	SHEET ADDED (CONT. MOD.)	E.J.M
	22 MAR 77	SHEET ADDED (CONT MOD)	D.L.T
4	NOV 76	SHEET ADDED (CONT MOD)	DB
	MAY 76	SHEET ADDED (CONT MOD)	E.J.M
2	DEC 75	INDEX REVISED (CONT. MOD.)	E.J.M
1	19 NOV 74	ADDED SHEETS 40 & 93 (AMDT NO 2)	

U. S. ARMY ENGINEER DISTRICT, LOUISVILLE
CORPS OF ENGINEERS
LOUISVILLE KENTUCKY

DESIGNED: RDP A.H.
DRAWN: J.E.N. TRACED: J.E.N. & J.M.W.
CHECKED: [illegible] R.R.
SUBMITTED: [illegible]
APPROVAL RECOM.
ASST CHIEF ENGINEERING DIV.

WABASH RIVER BASIN
PATOKA LAKE
PATOKA RIVER, IND
DAM & SPILLWAY
LOCATION PLAN, VICINITY MAP & INDEX

APPROVED: Charles J. Fiala
COL. CORPS OF ENGINEERS, DISTRICT ENGINEER
DATE: NOV '74

APPROVED: R.E. Karlen
CHIEF ENGINEERING DIVISION

SCALE: AS SHOWN

DRAWING NUMBER: **PR 18-12.5/1**

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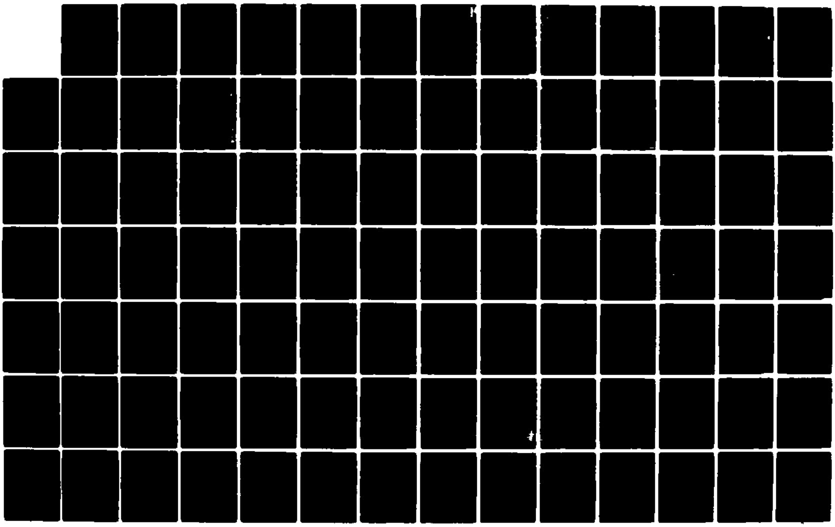
1

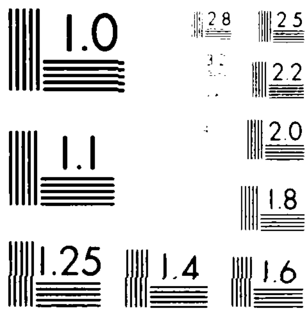
6

AD-A127 934 PATOKA LAKE FOUNDATION REPORT BOOK 2 BASIC REPORT SECTIONS 9-13(U) ARMY ENGINEER DISTRICT LOUISVILLE KY S BARTLETT ET AL. APR 83

2/3

UNCLASSIFIED F/G 13/13 NL

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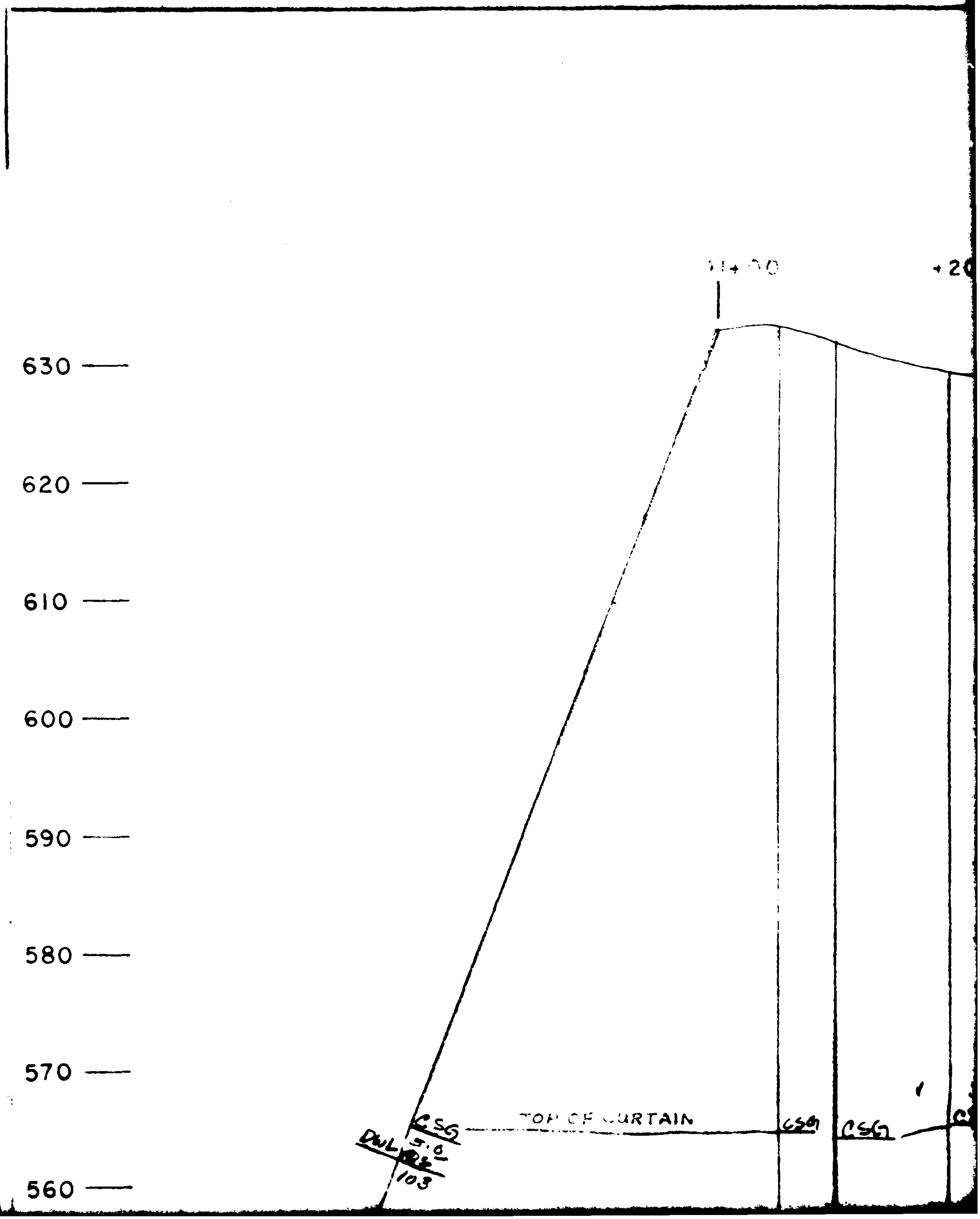
1.6

APPENDIX C

GROUT PROFILE DAM AND SPILLWAY TO
PATOKA LAKE FOUNDATION REPORT

Table of Contents



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TOP OF CURTAIN
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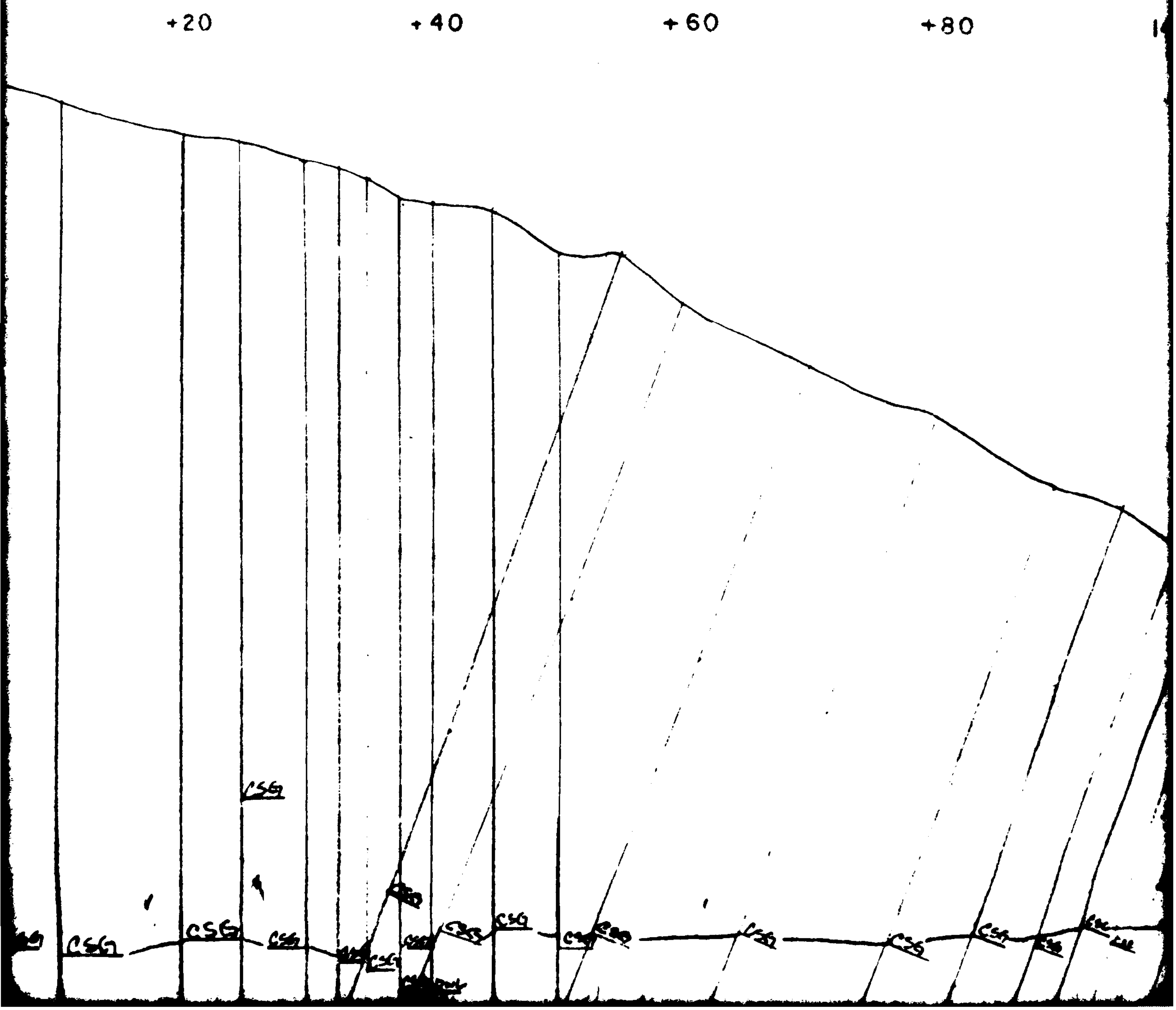
DWL
103
CSG
CSG



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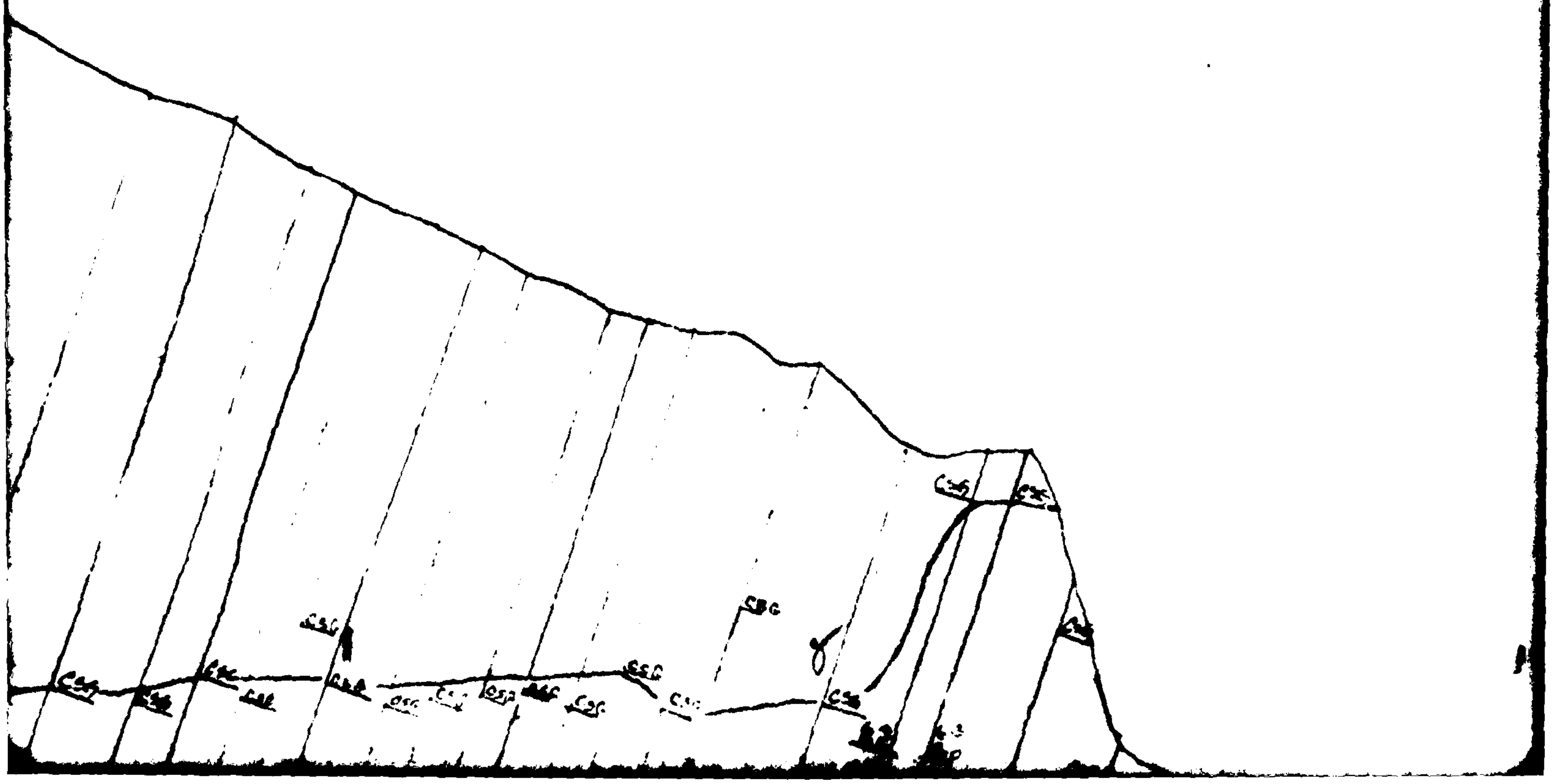

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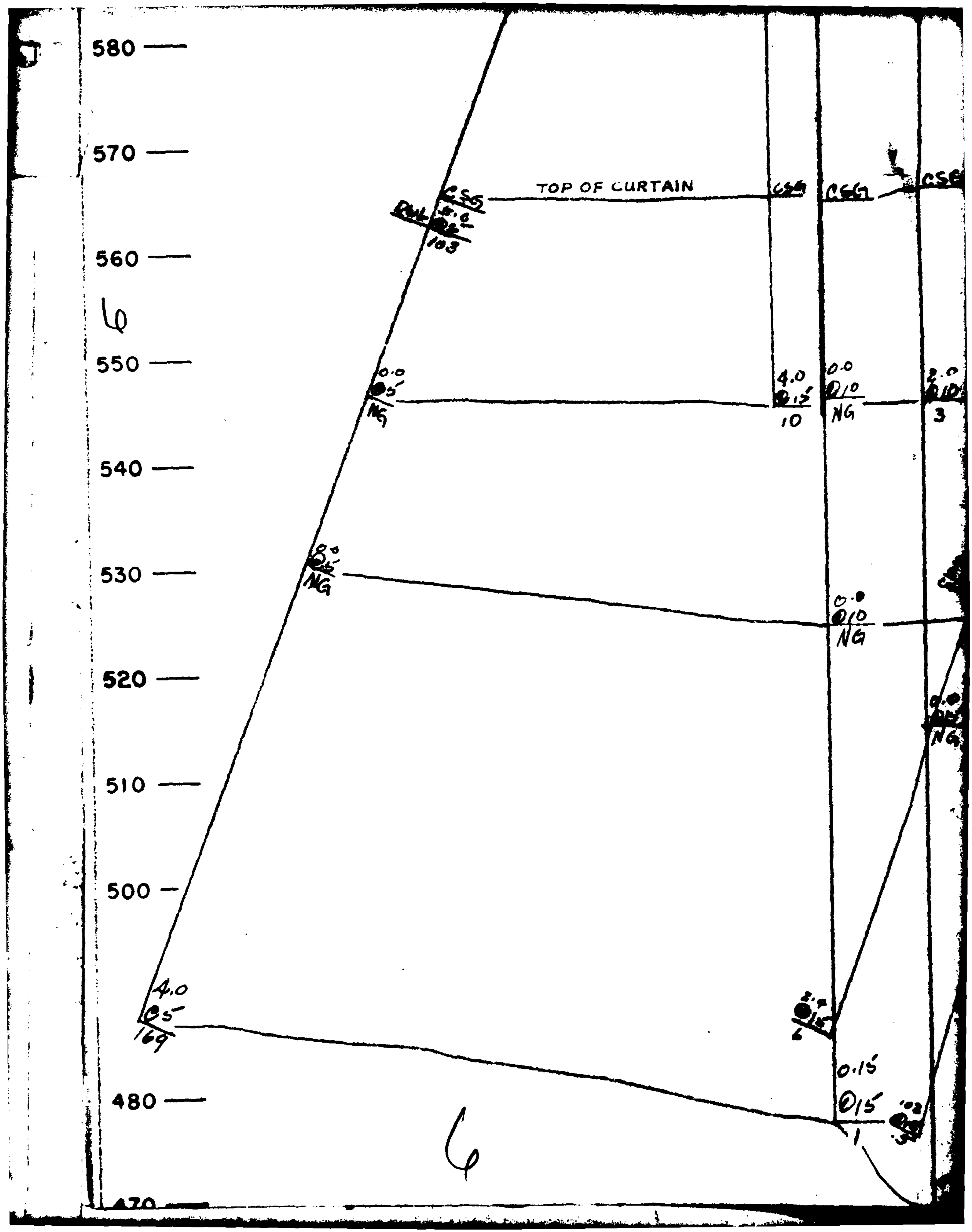
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TOP OF CURTAIN
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CSG
CSG
CSG
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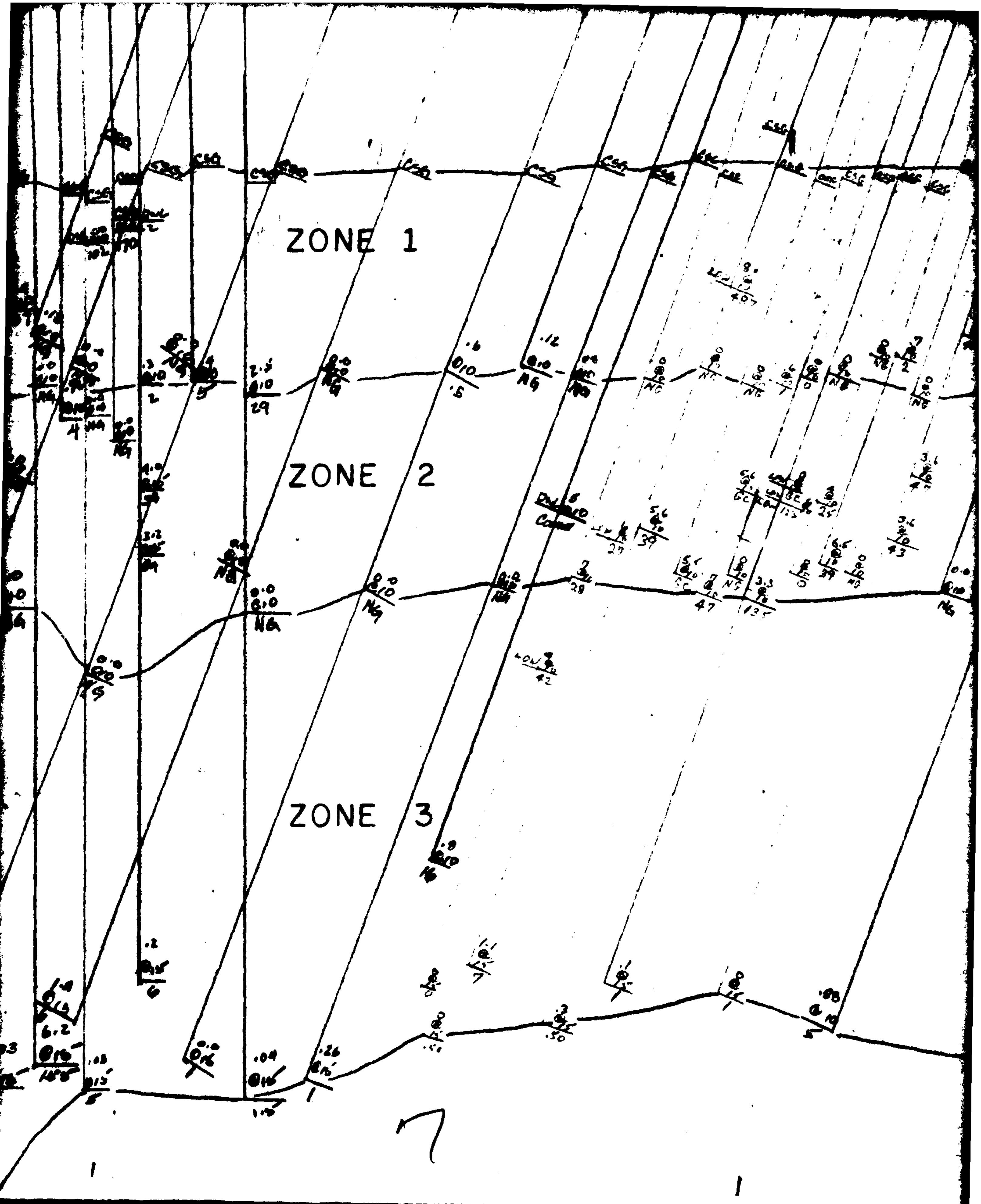

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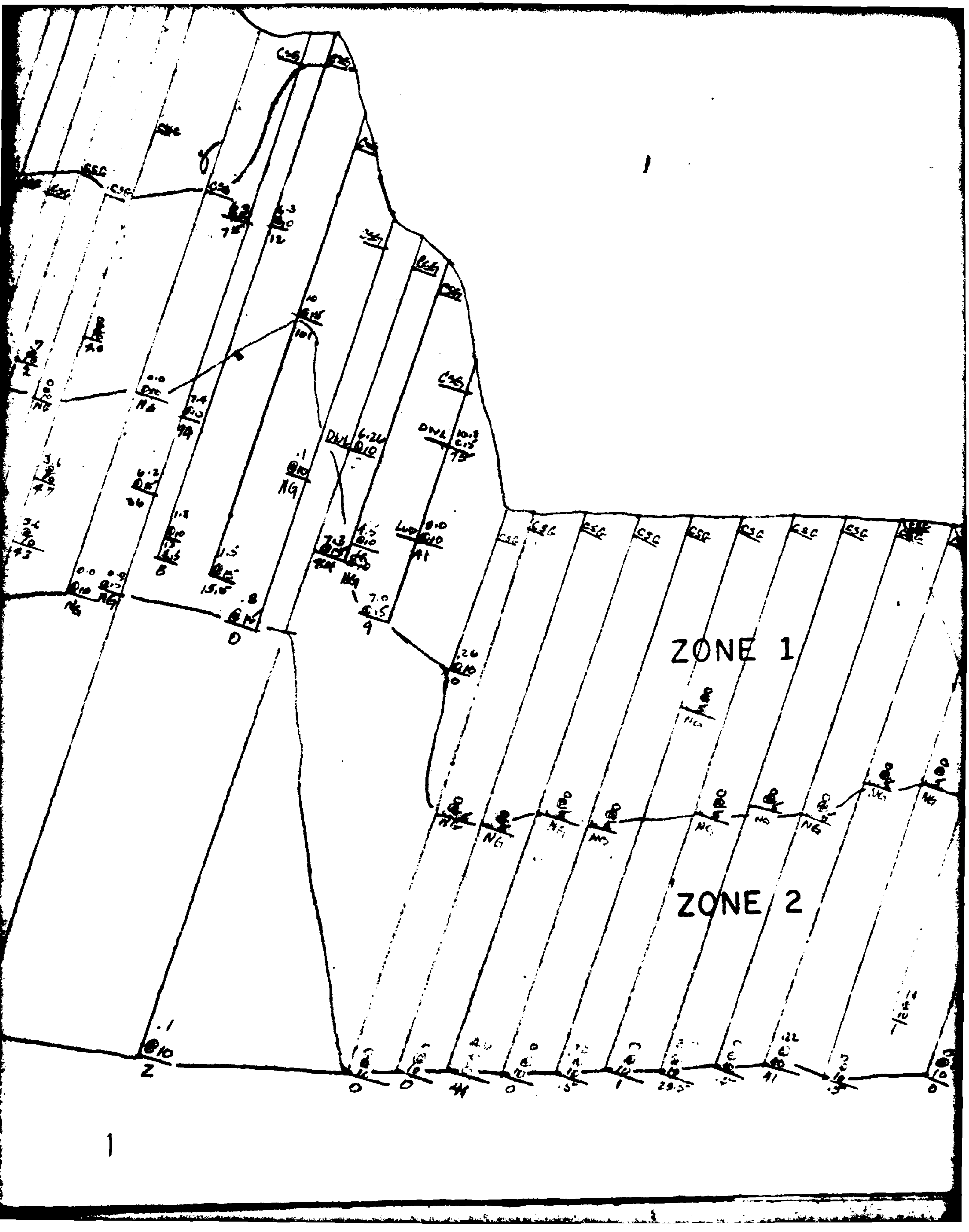
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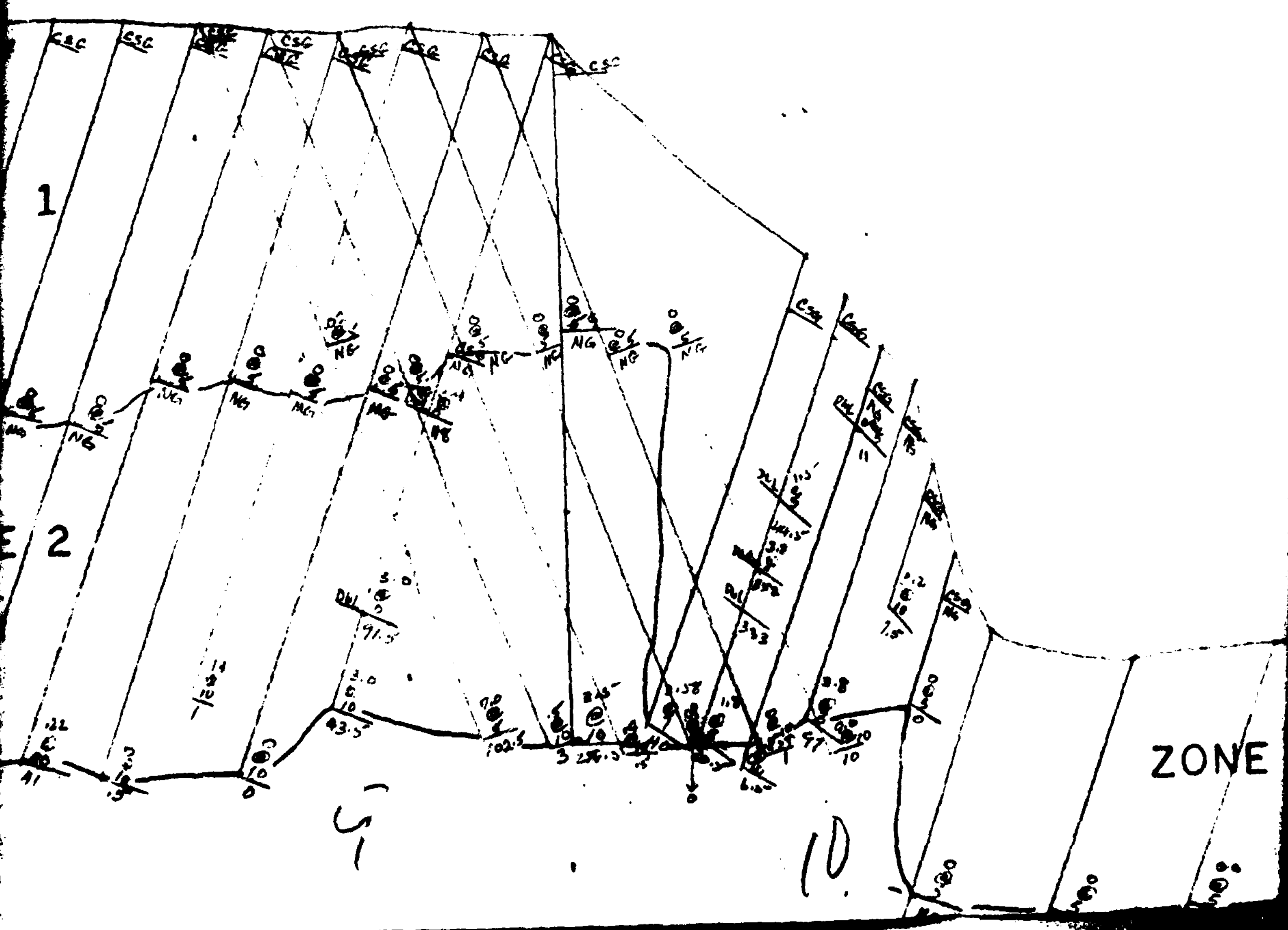

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E 2
ZONE
CSG
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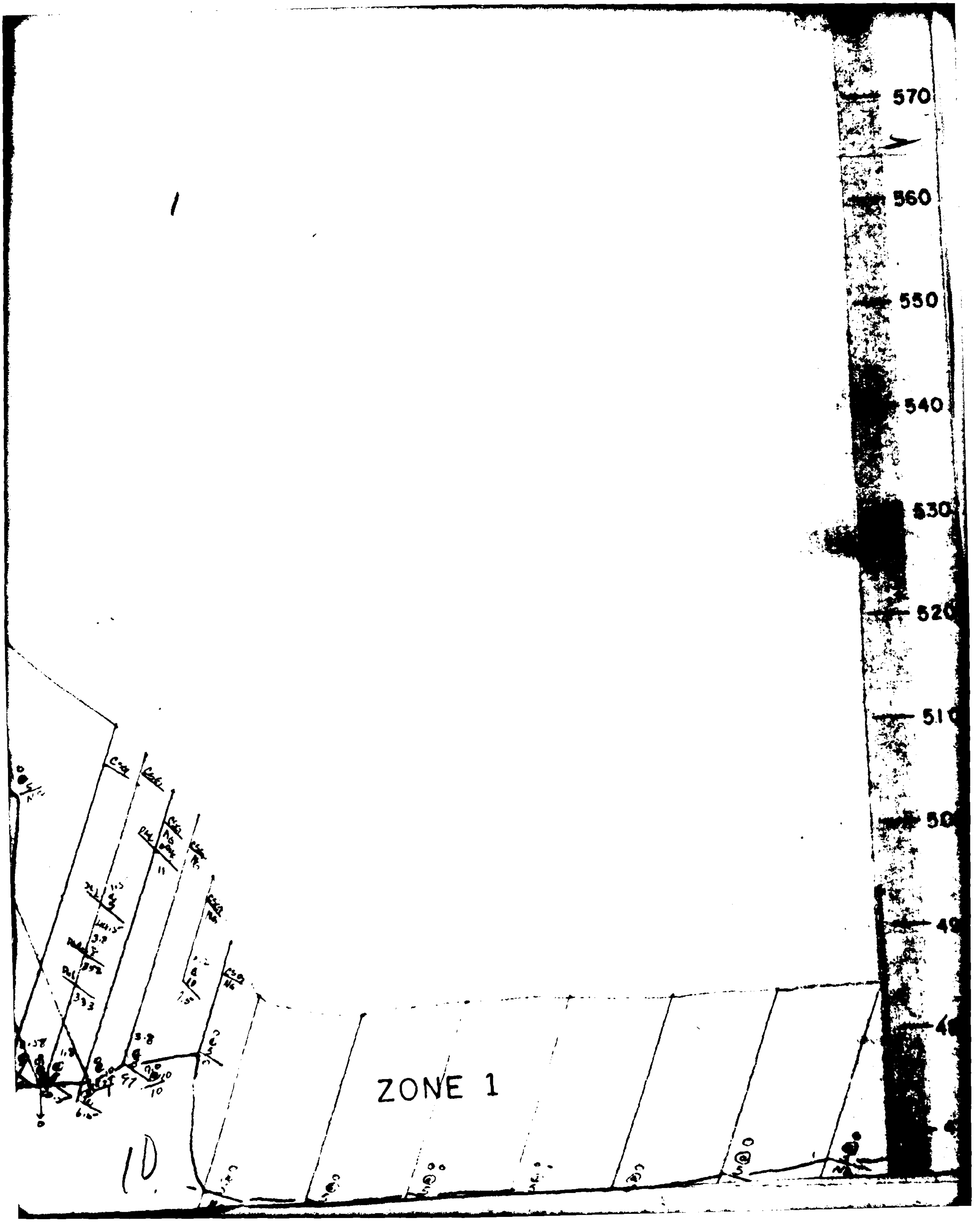
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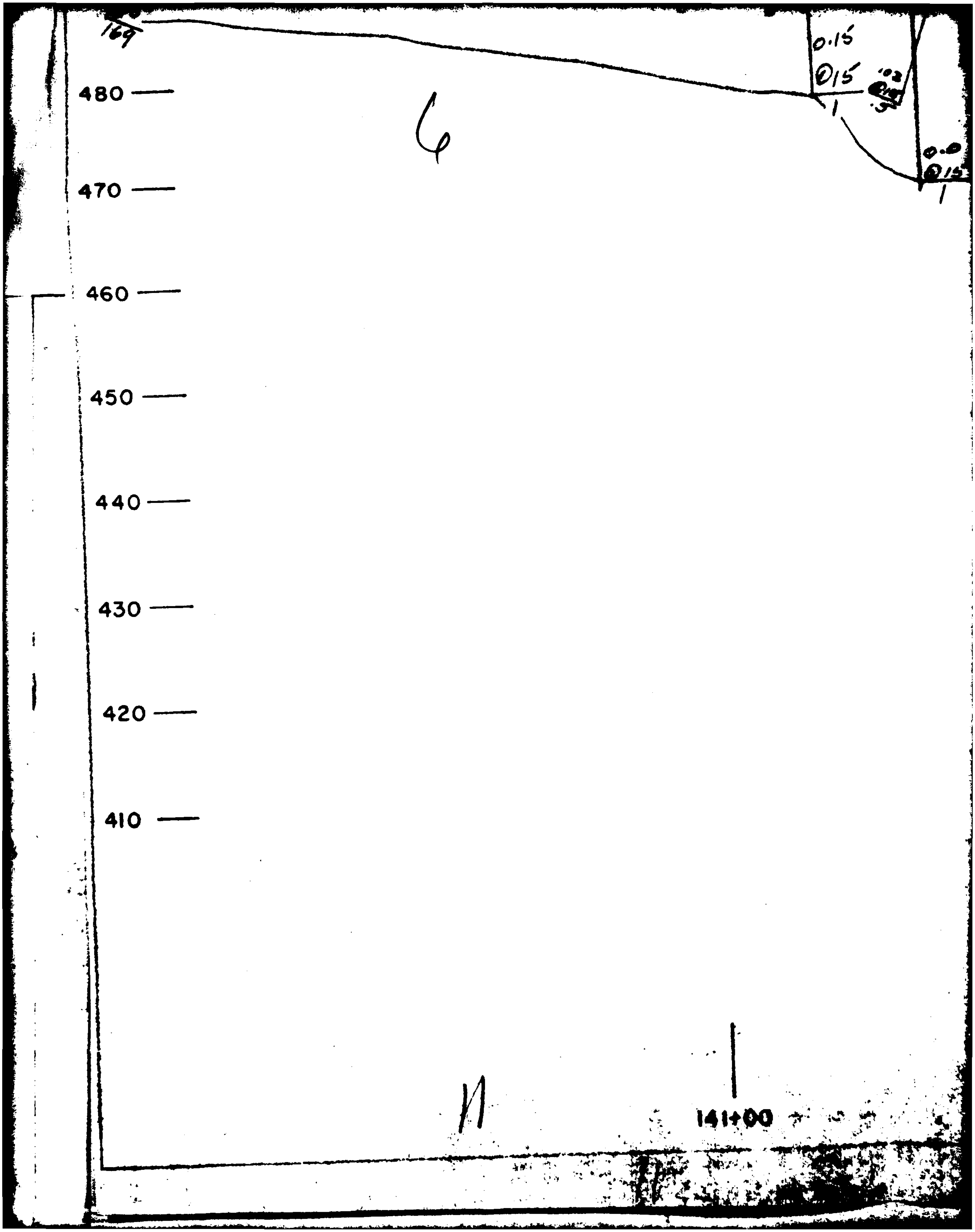

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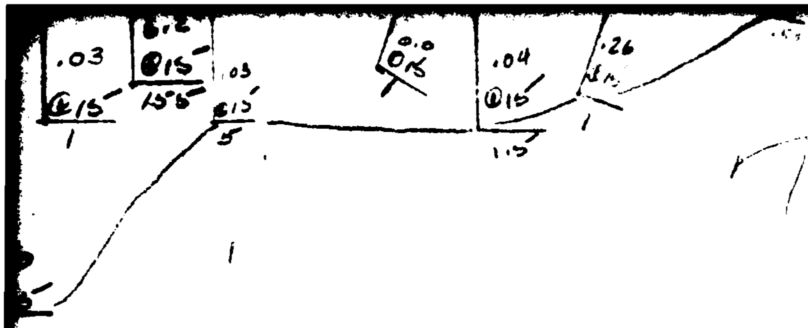

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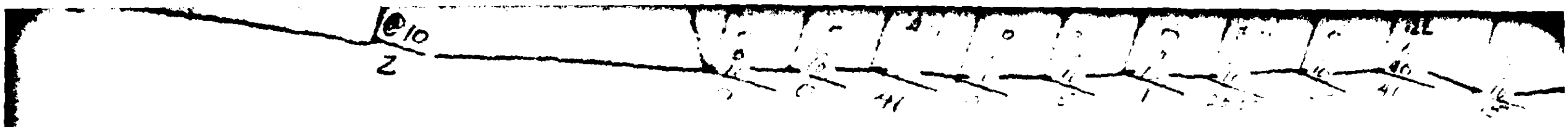
12



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13

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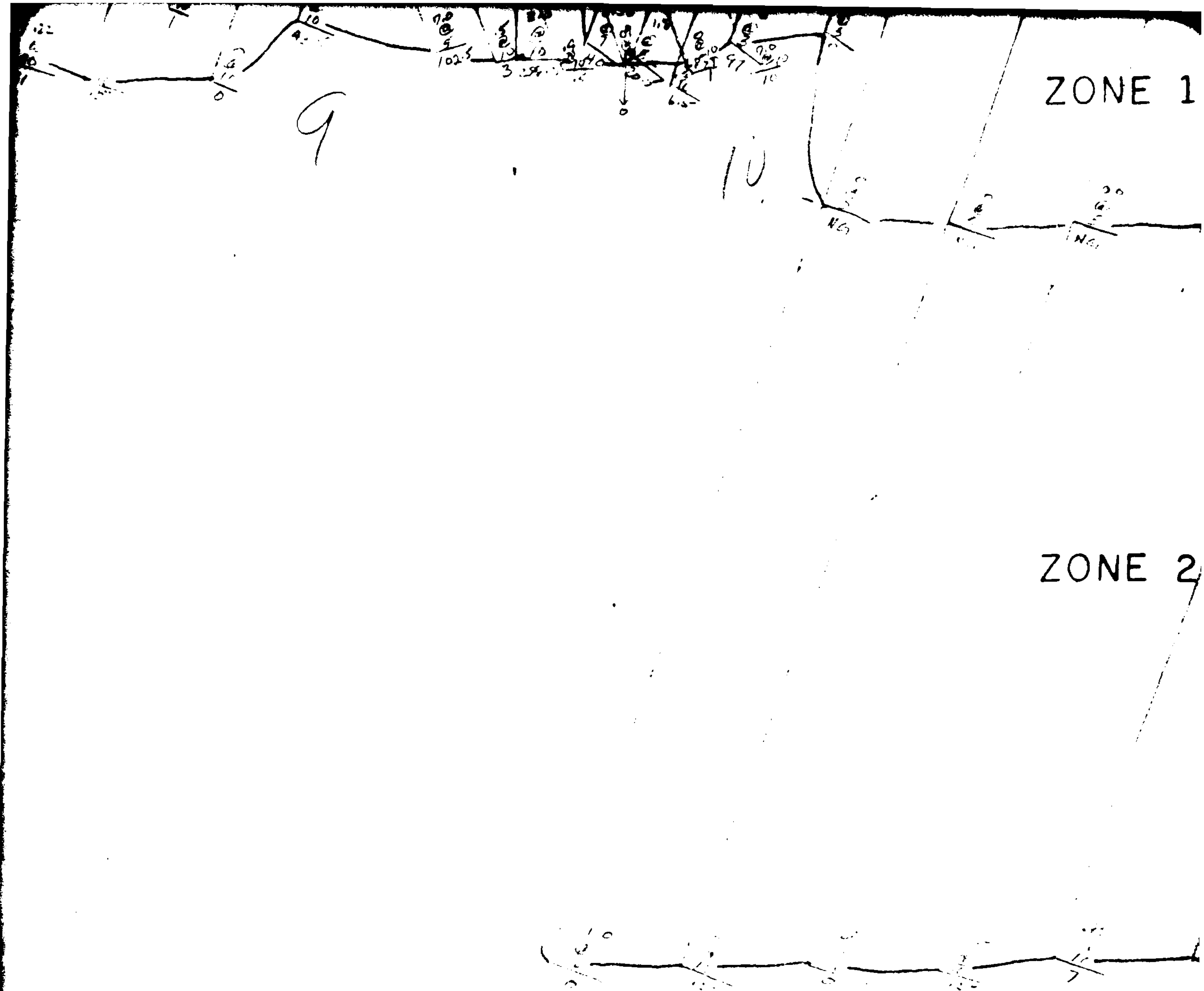

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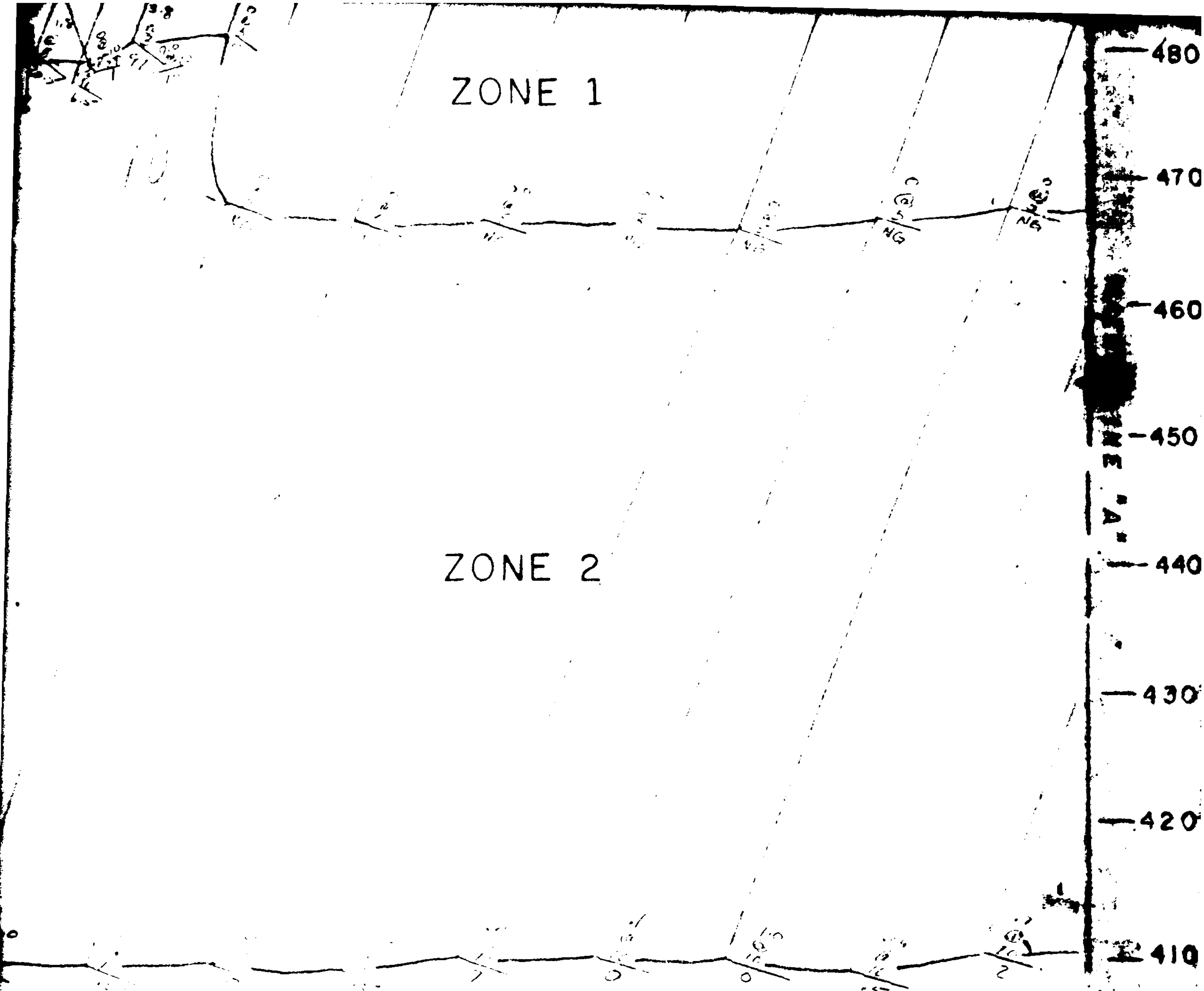

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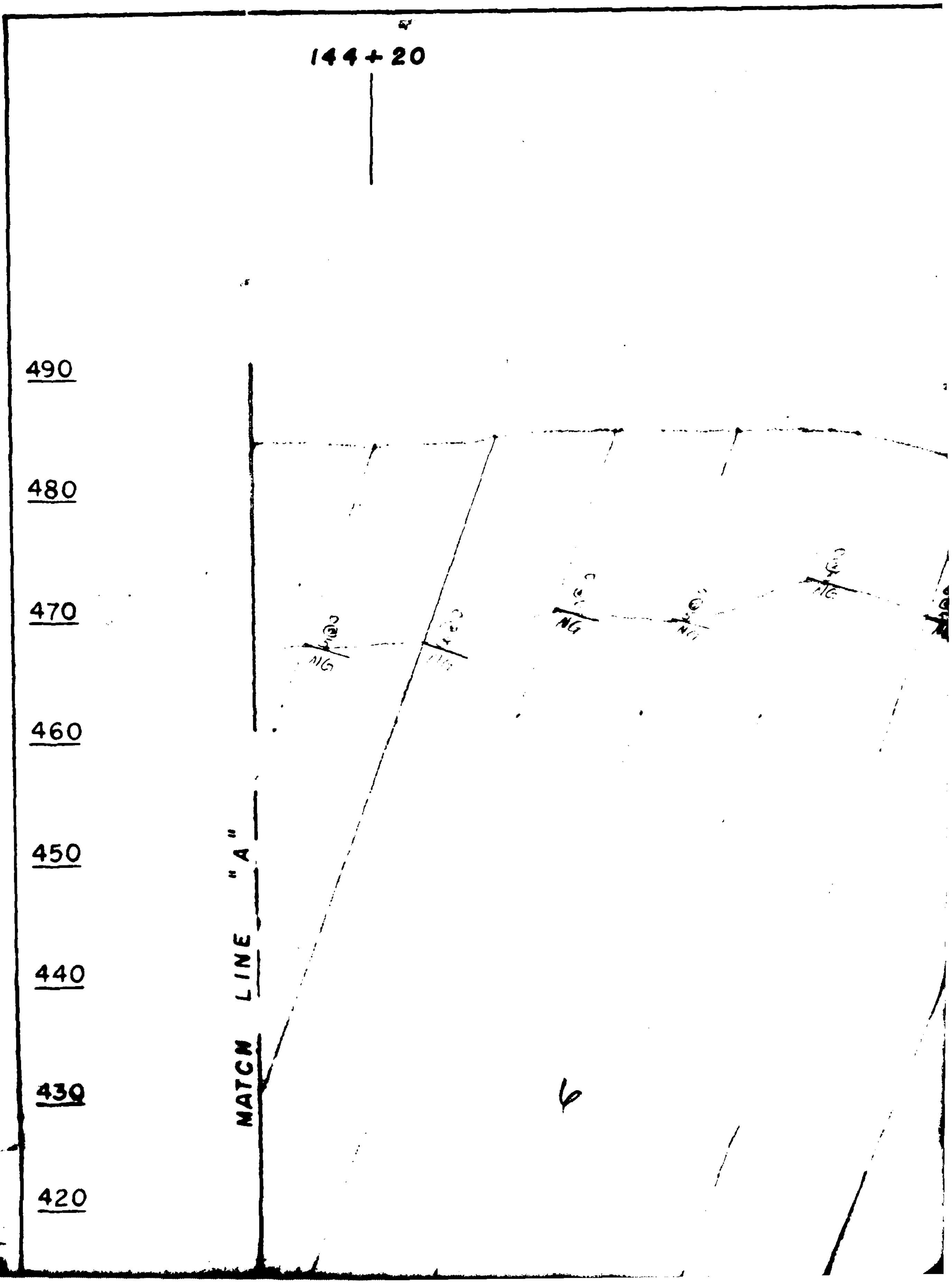
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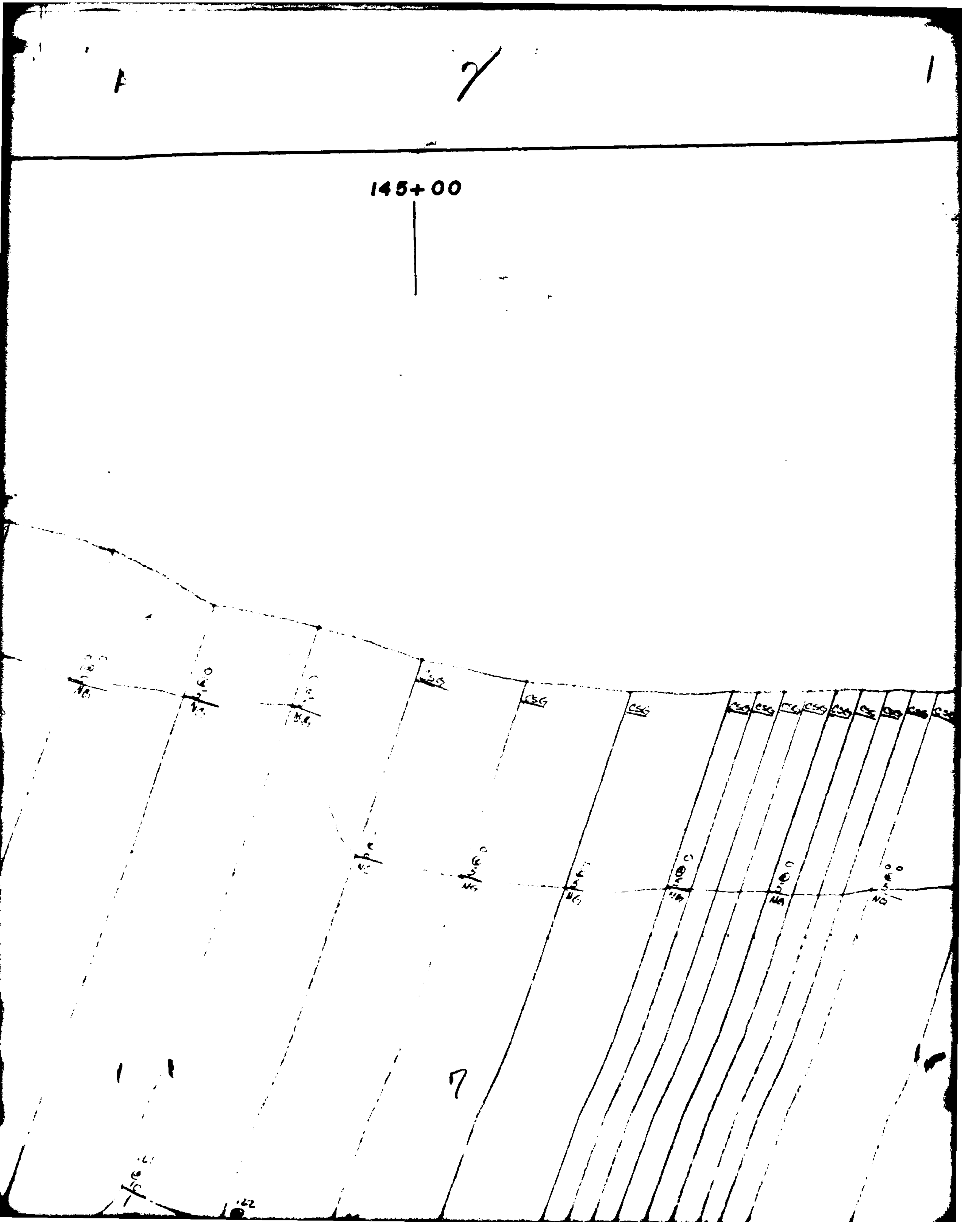
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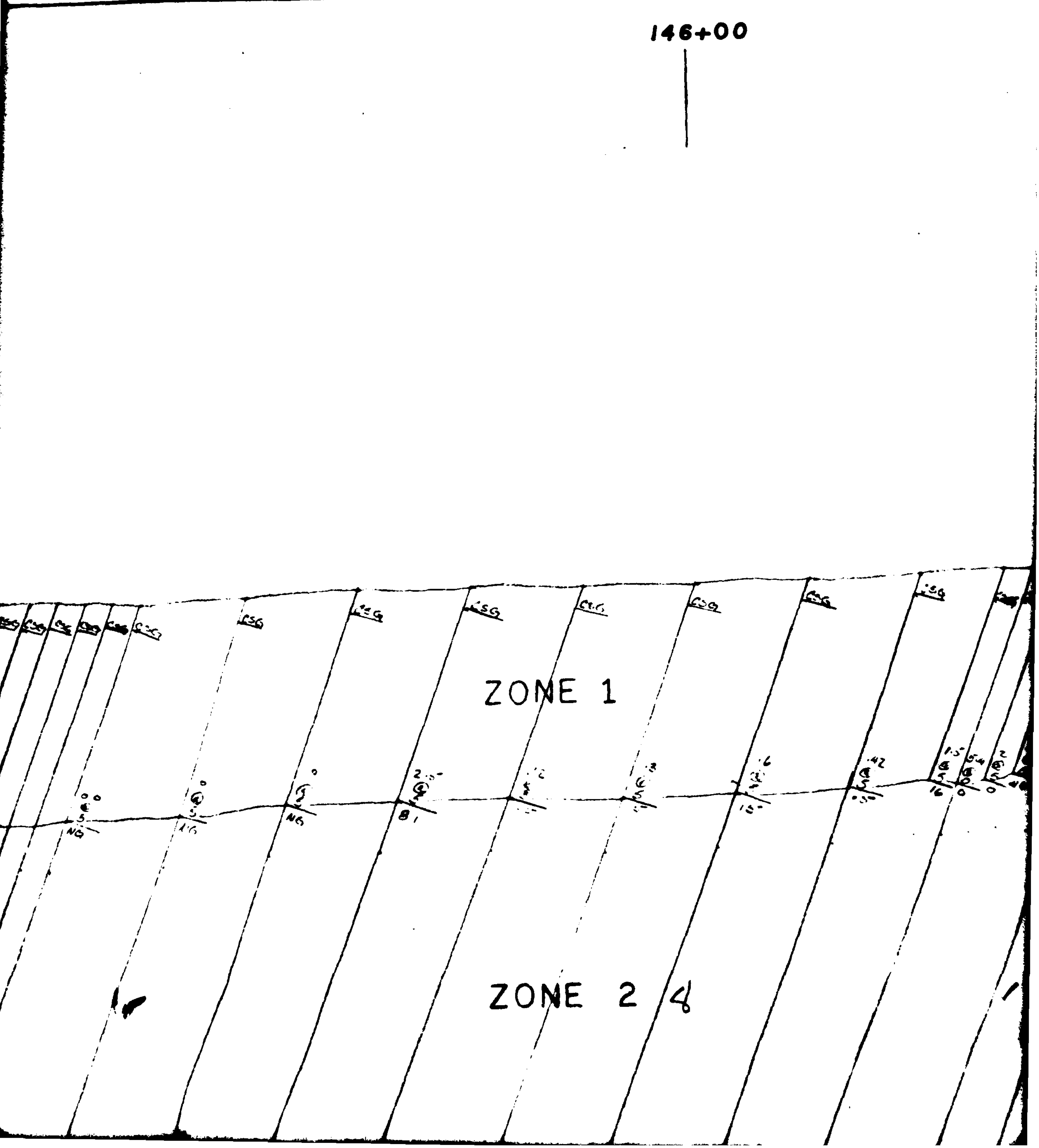

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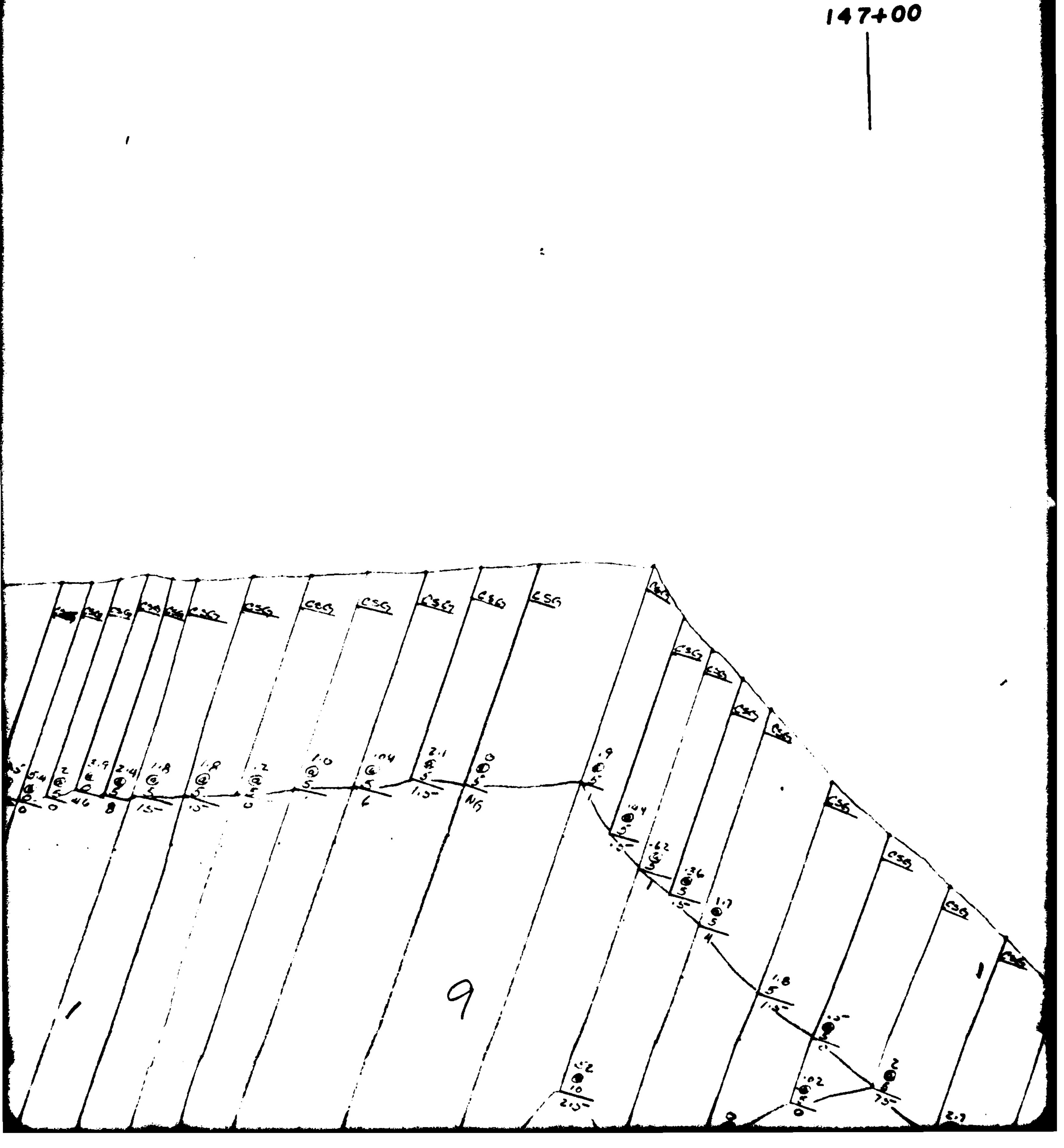

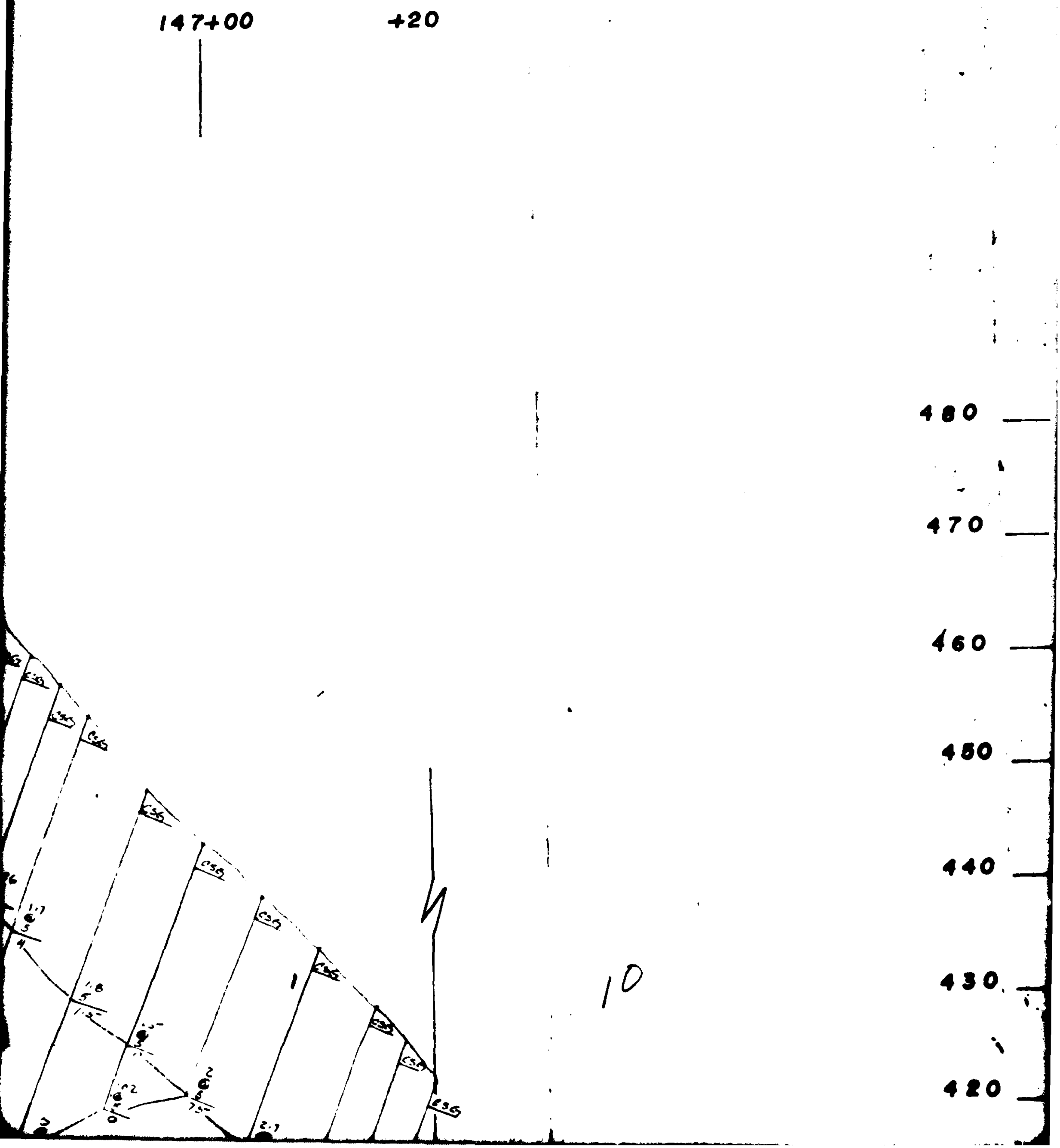

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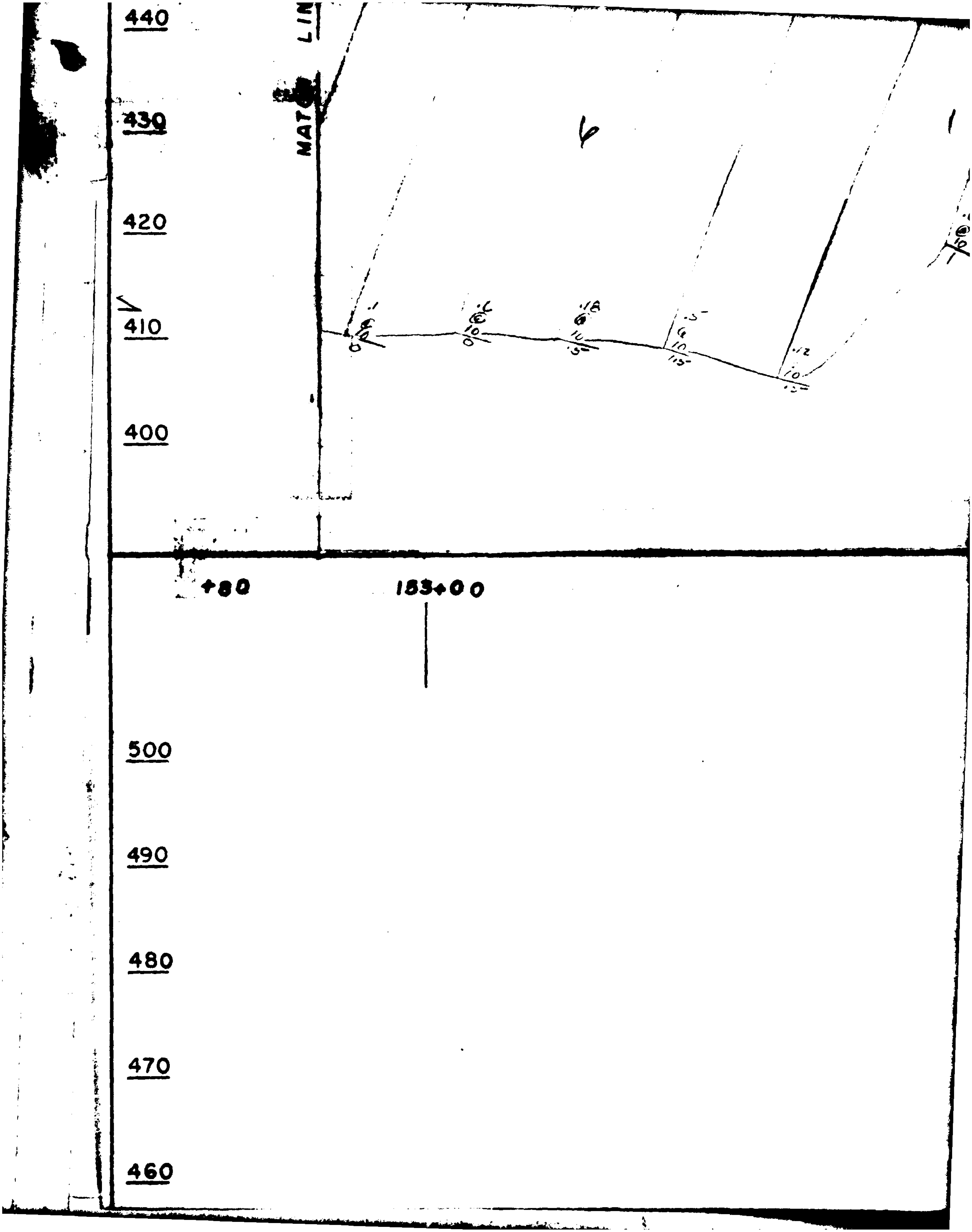

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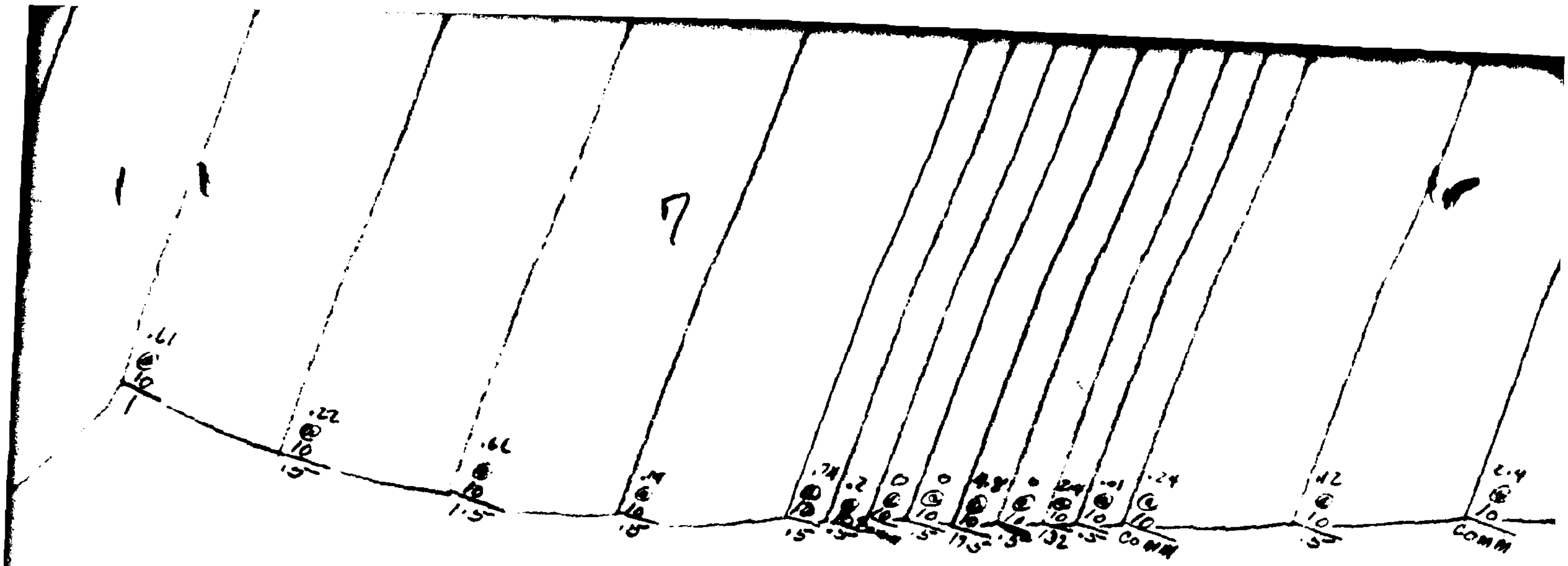

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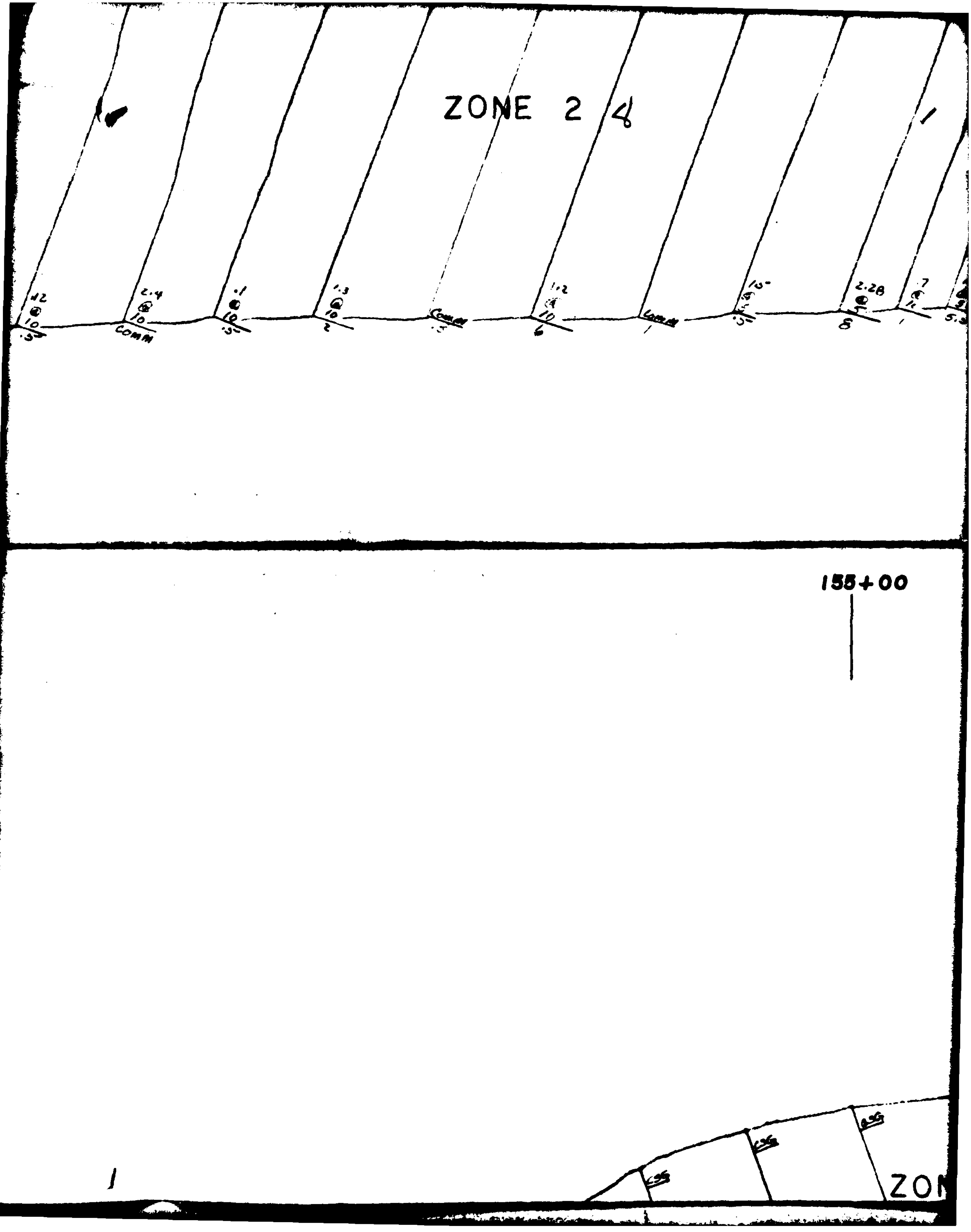
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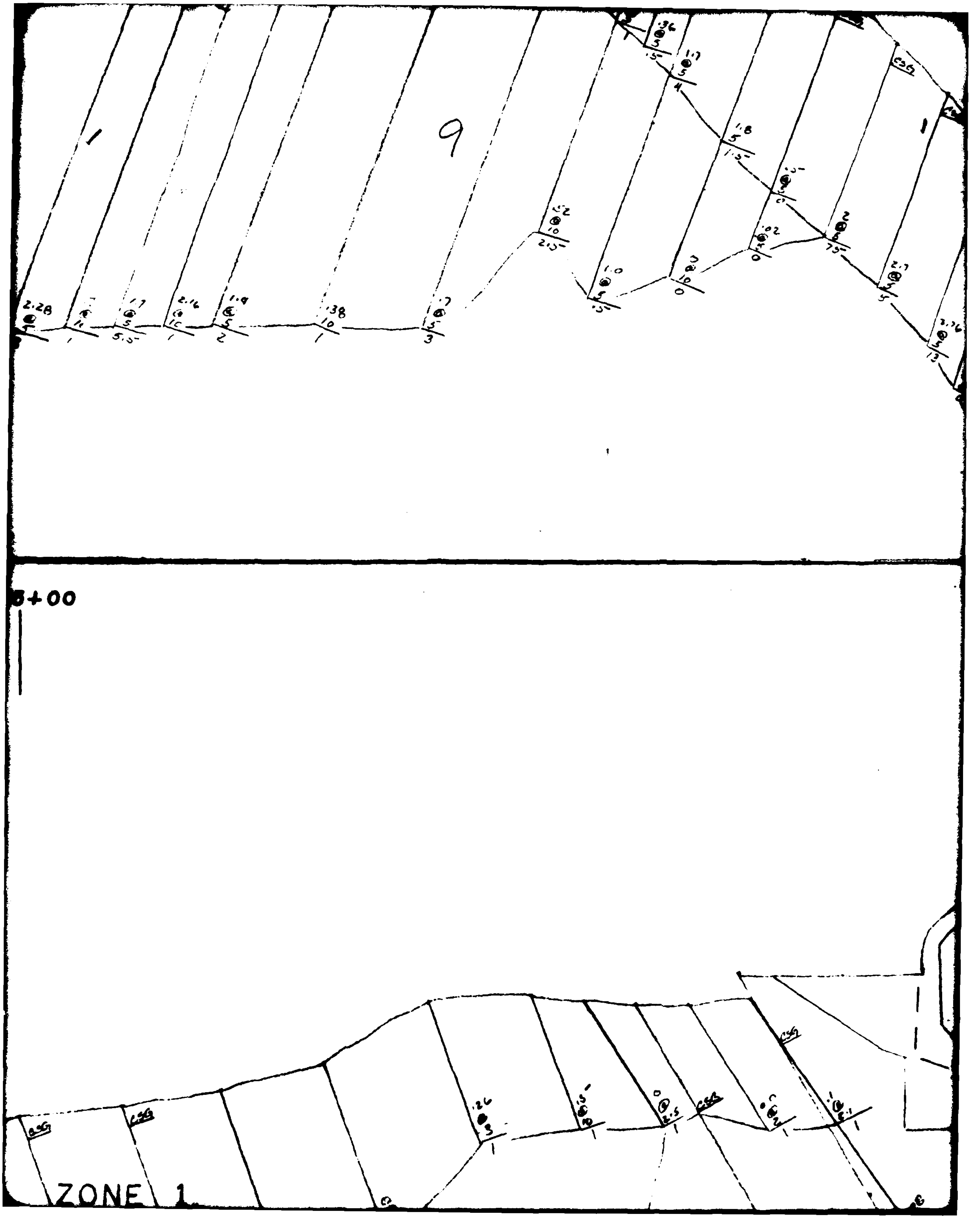
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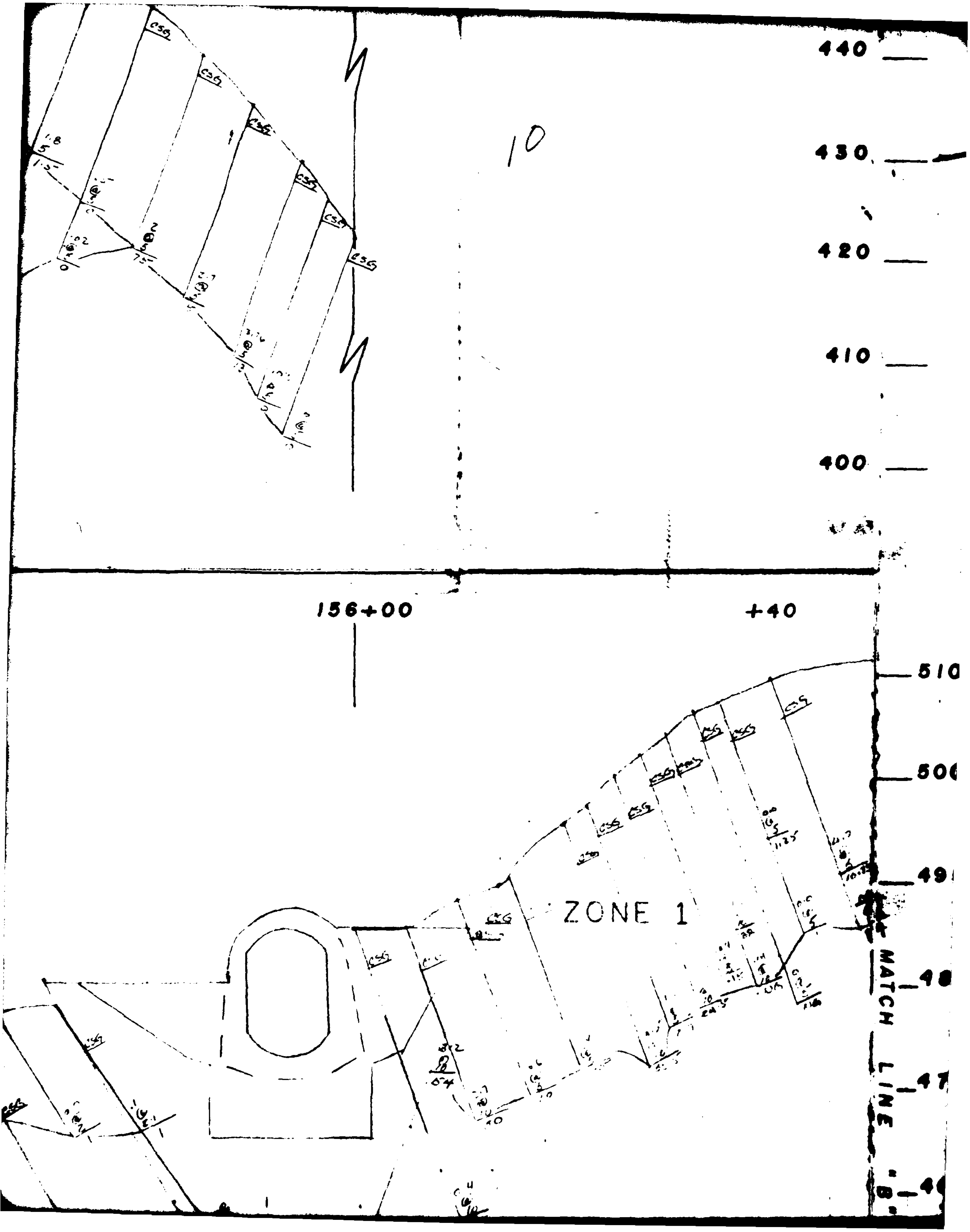
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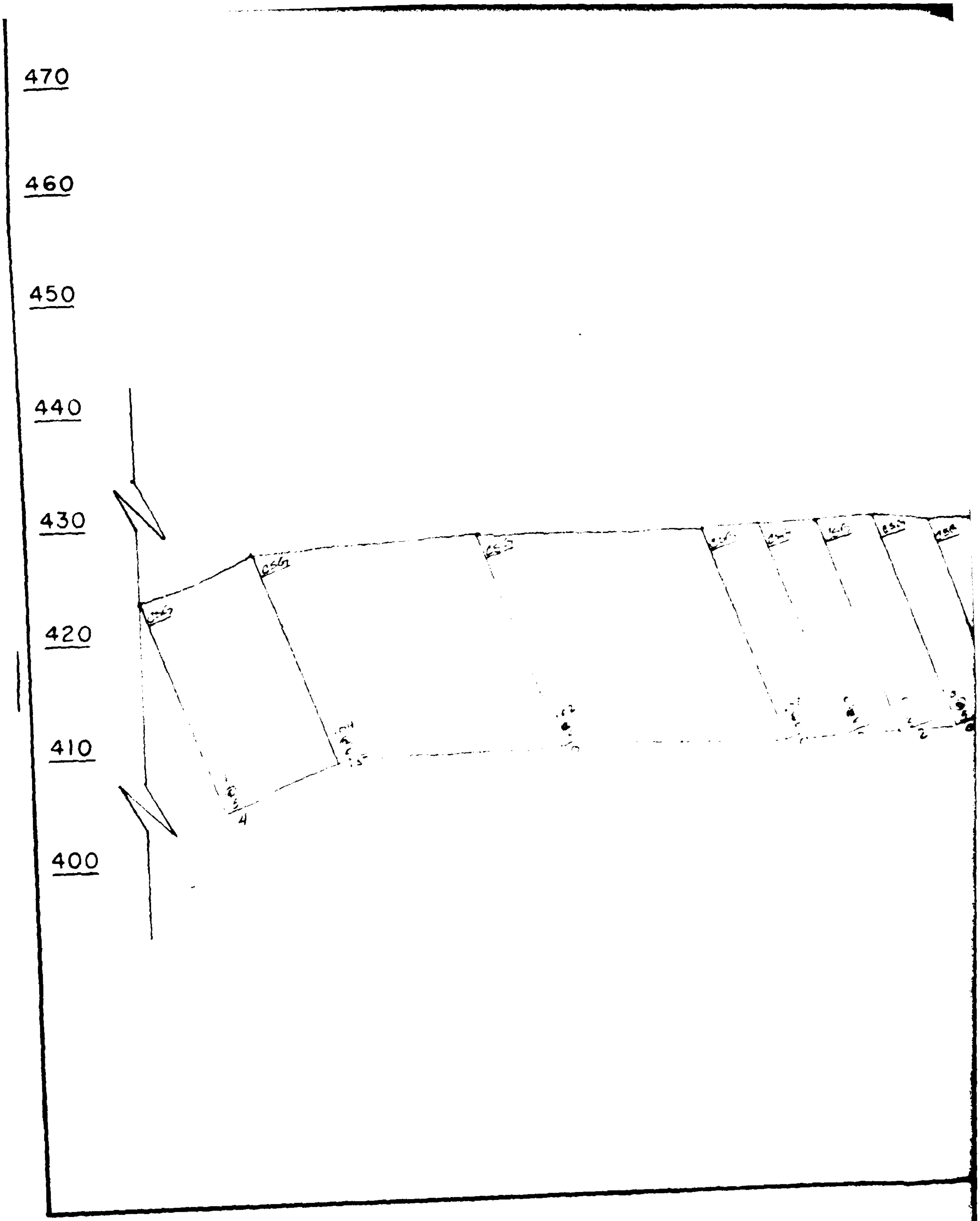
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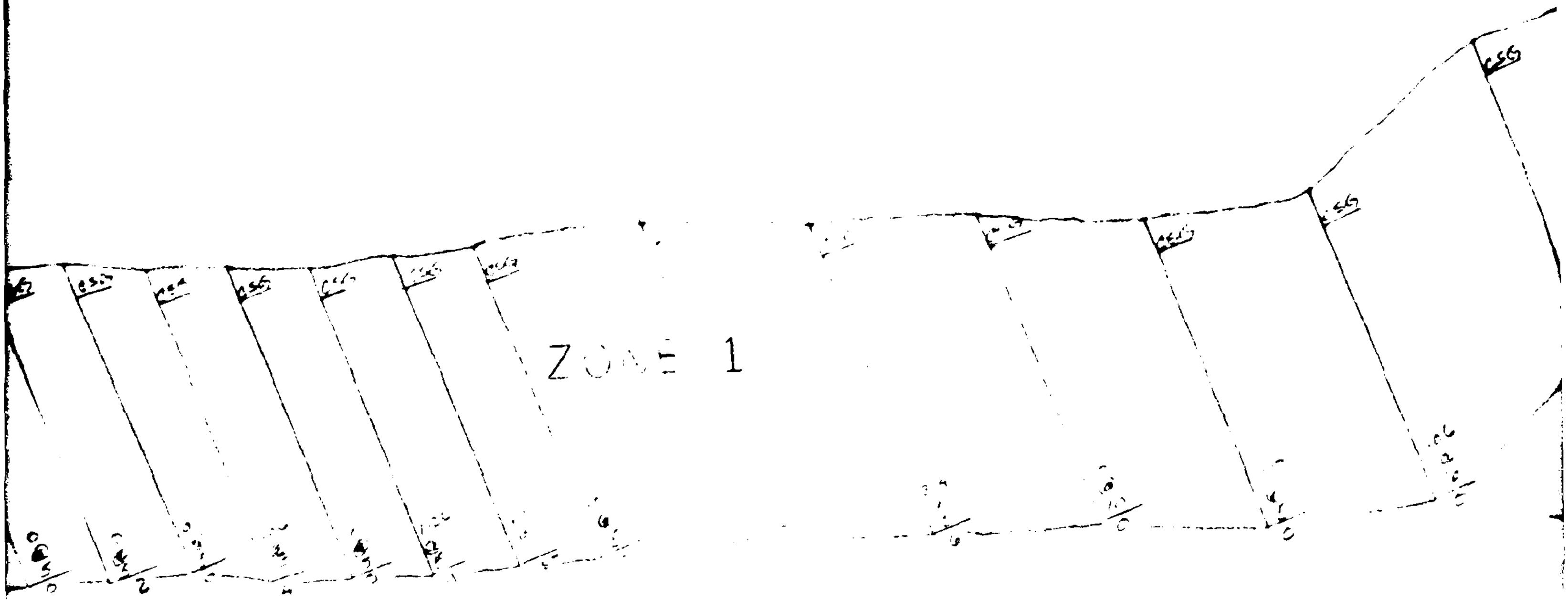
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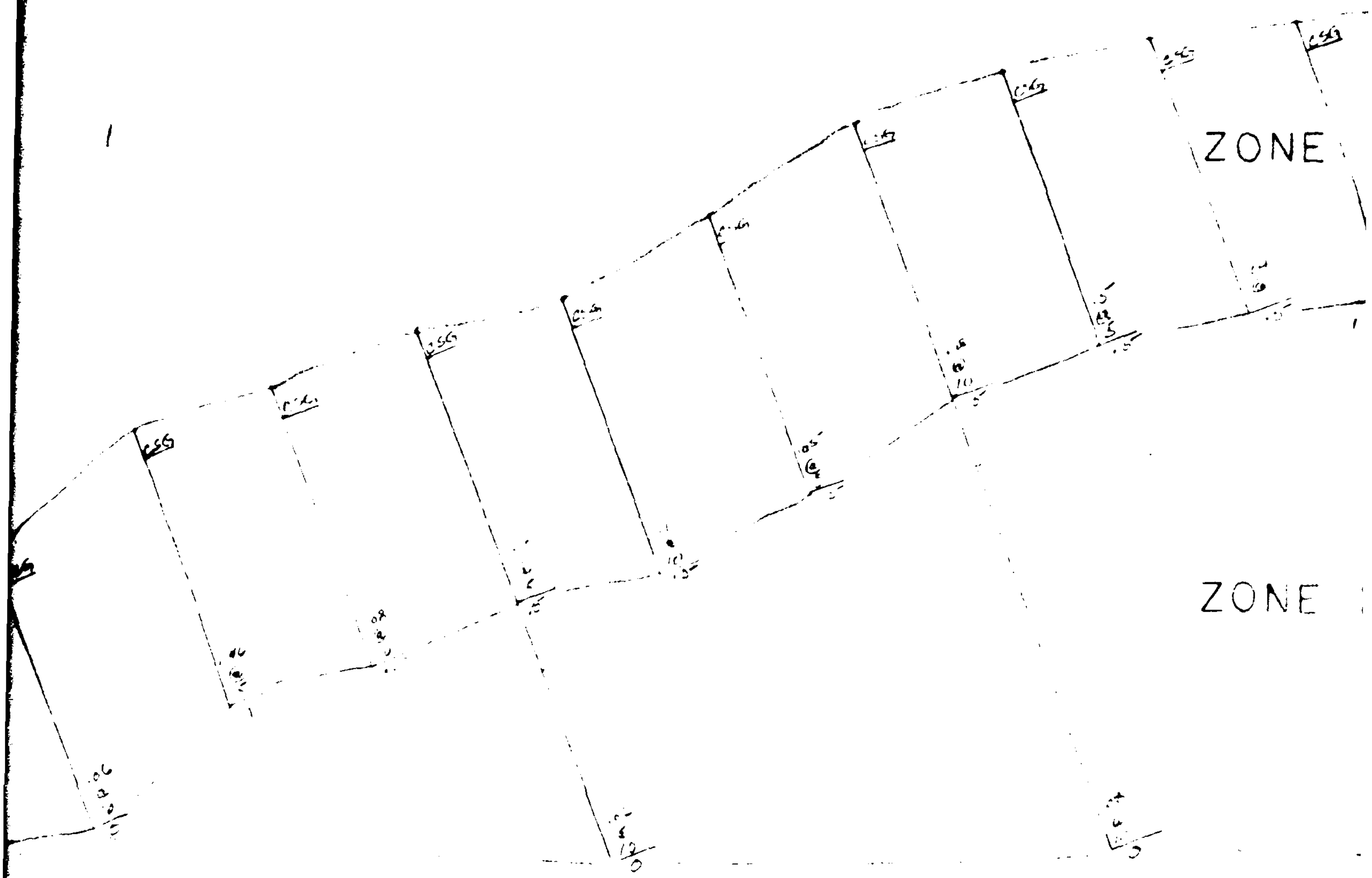
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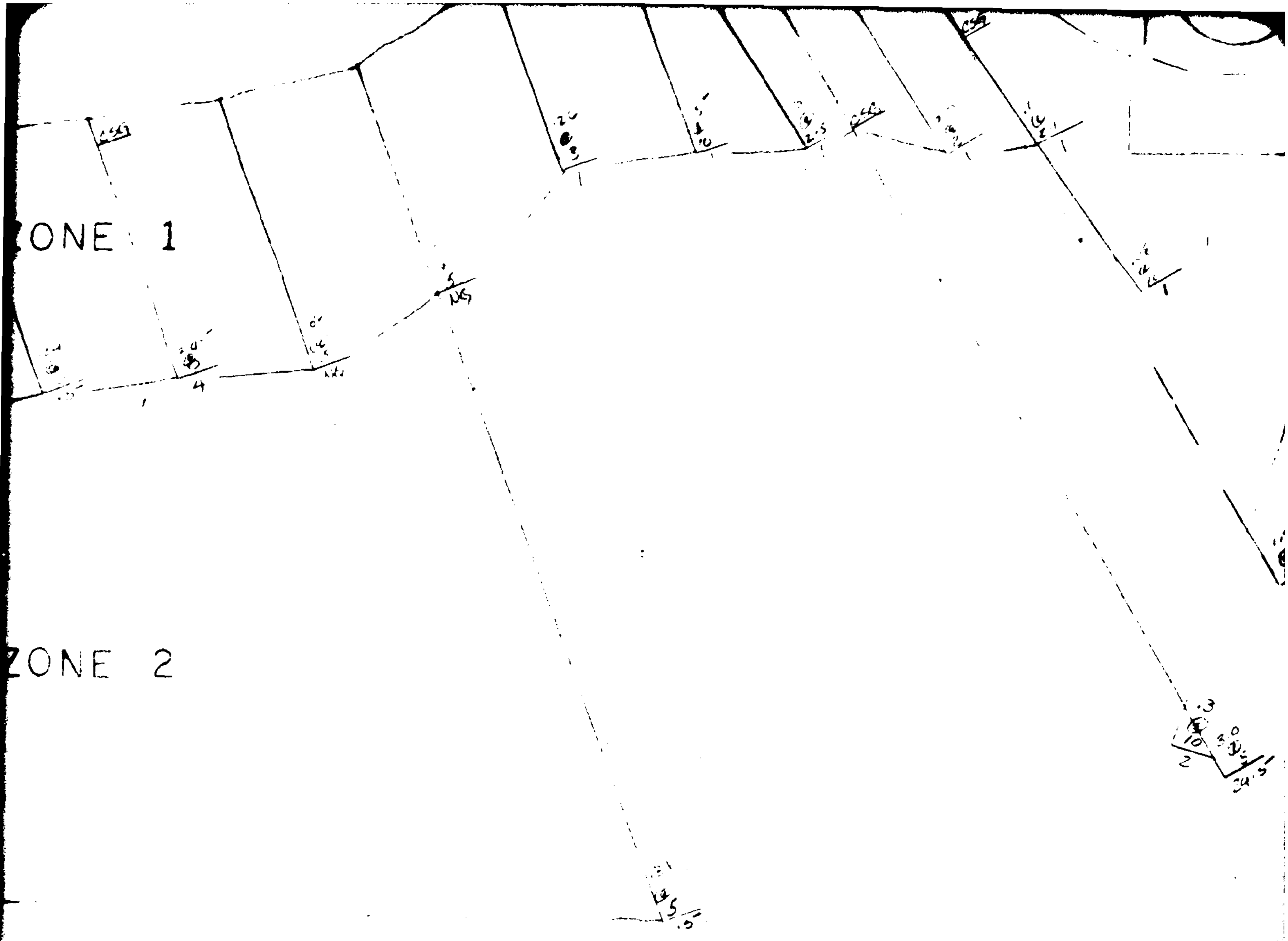

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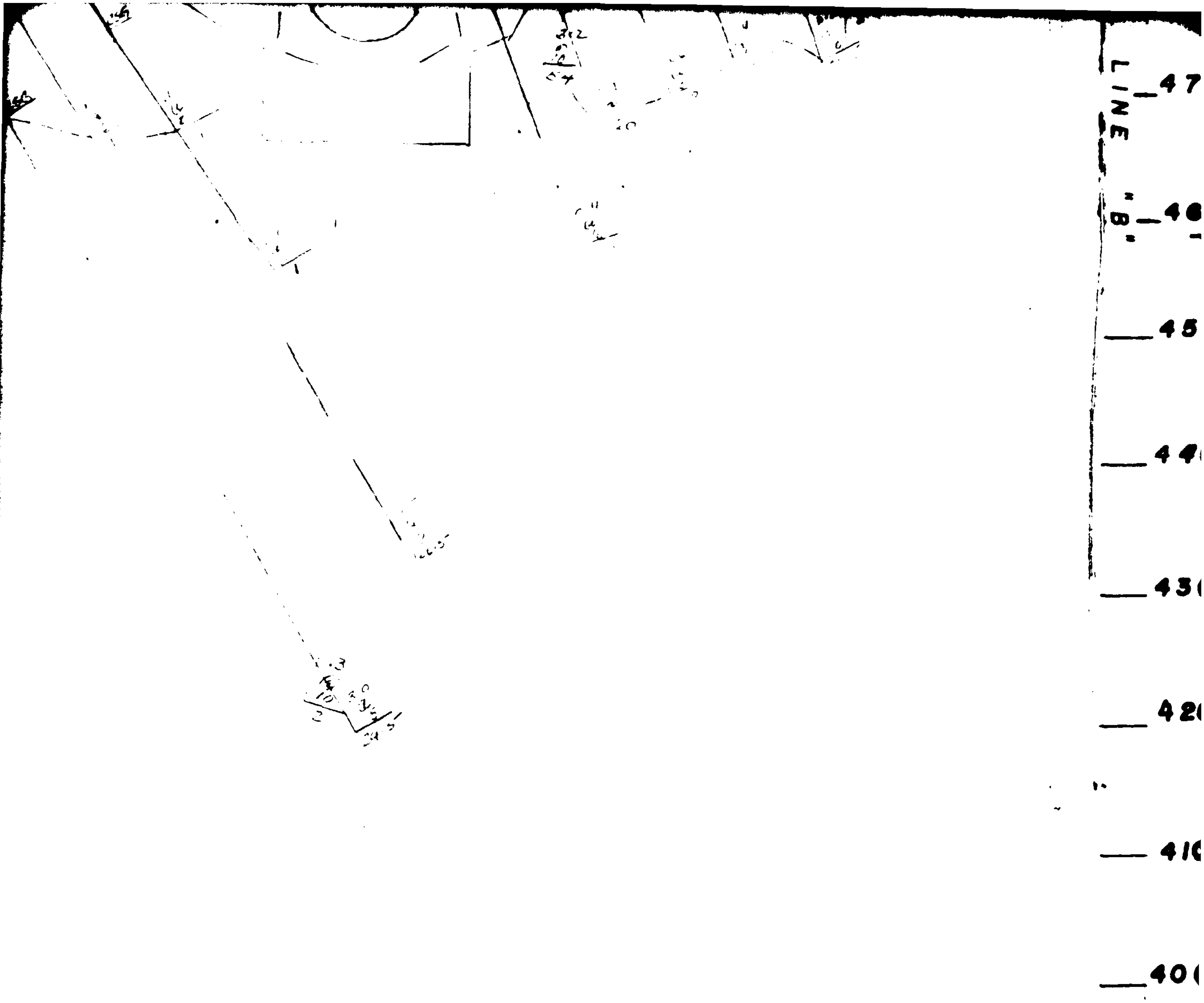

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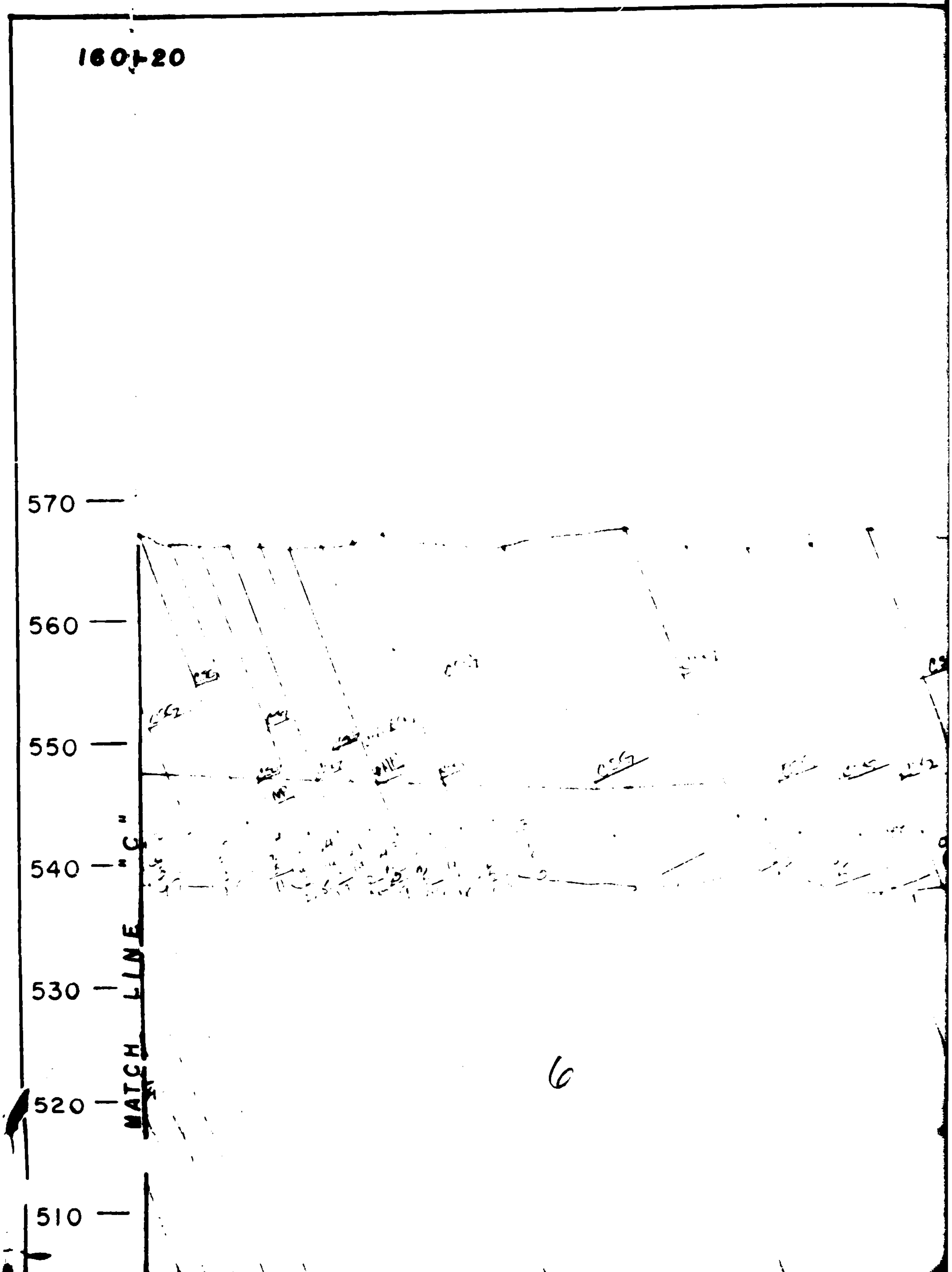
		DRAWING NUMBER FIGURE C-2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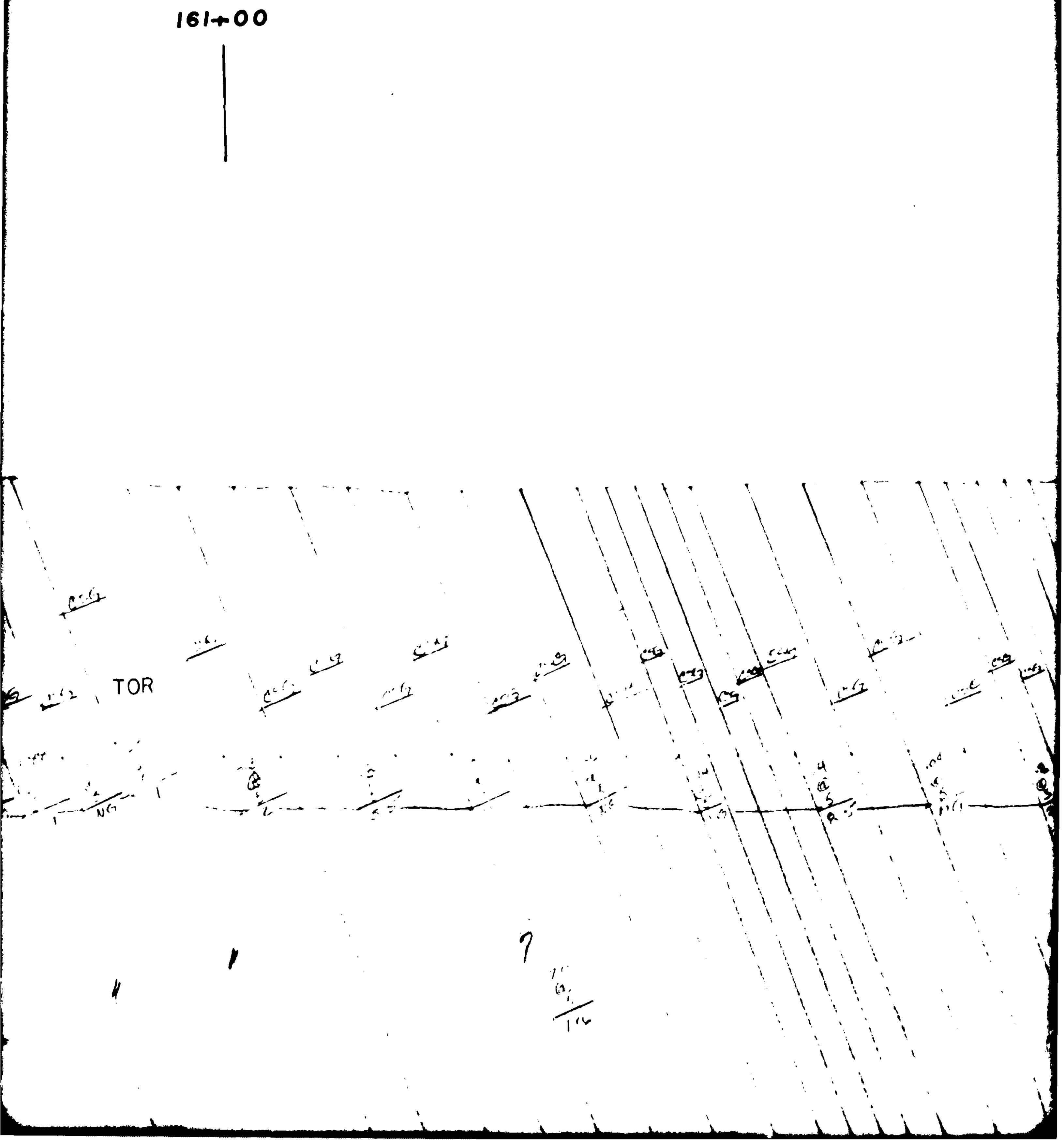
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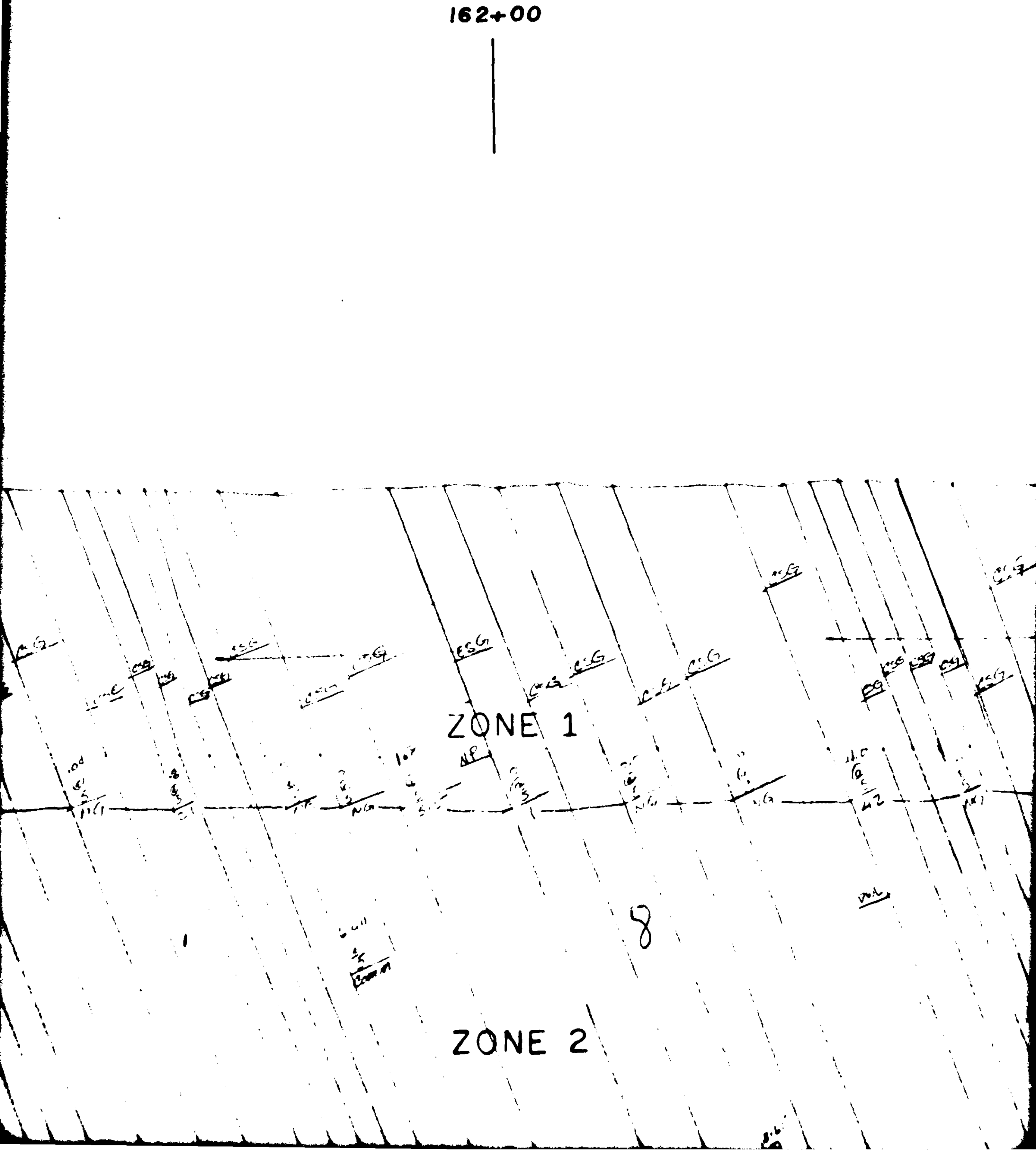

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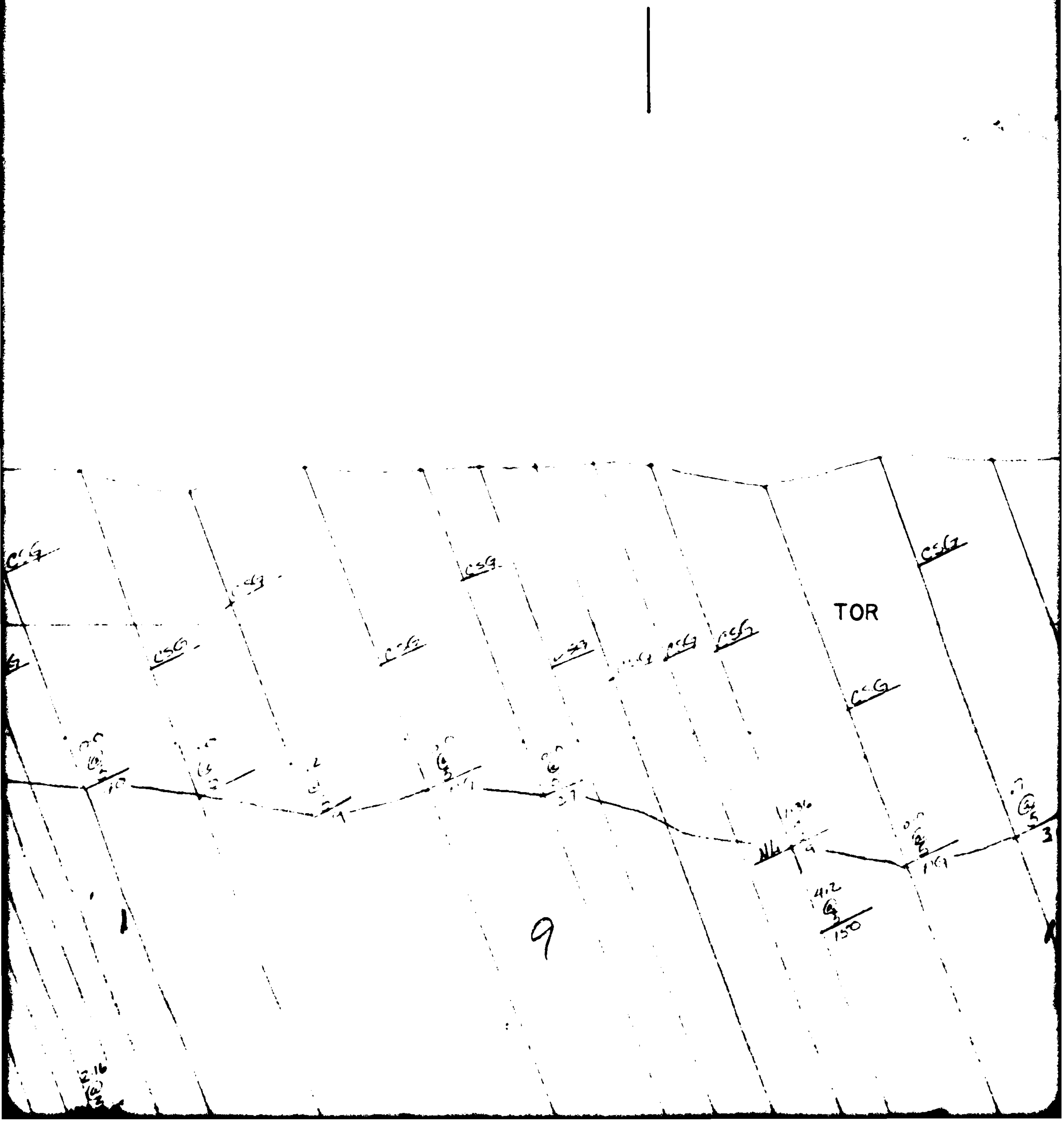
163+00



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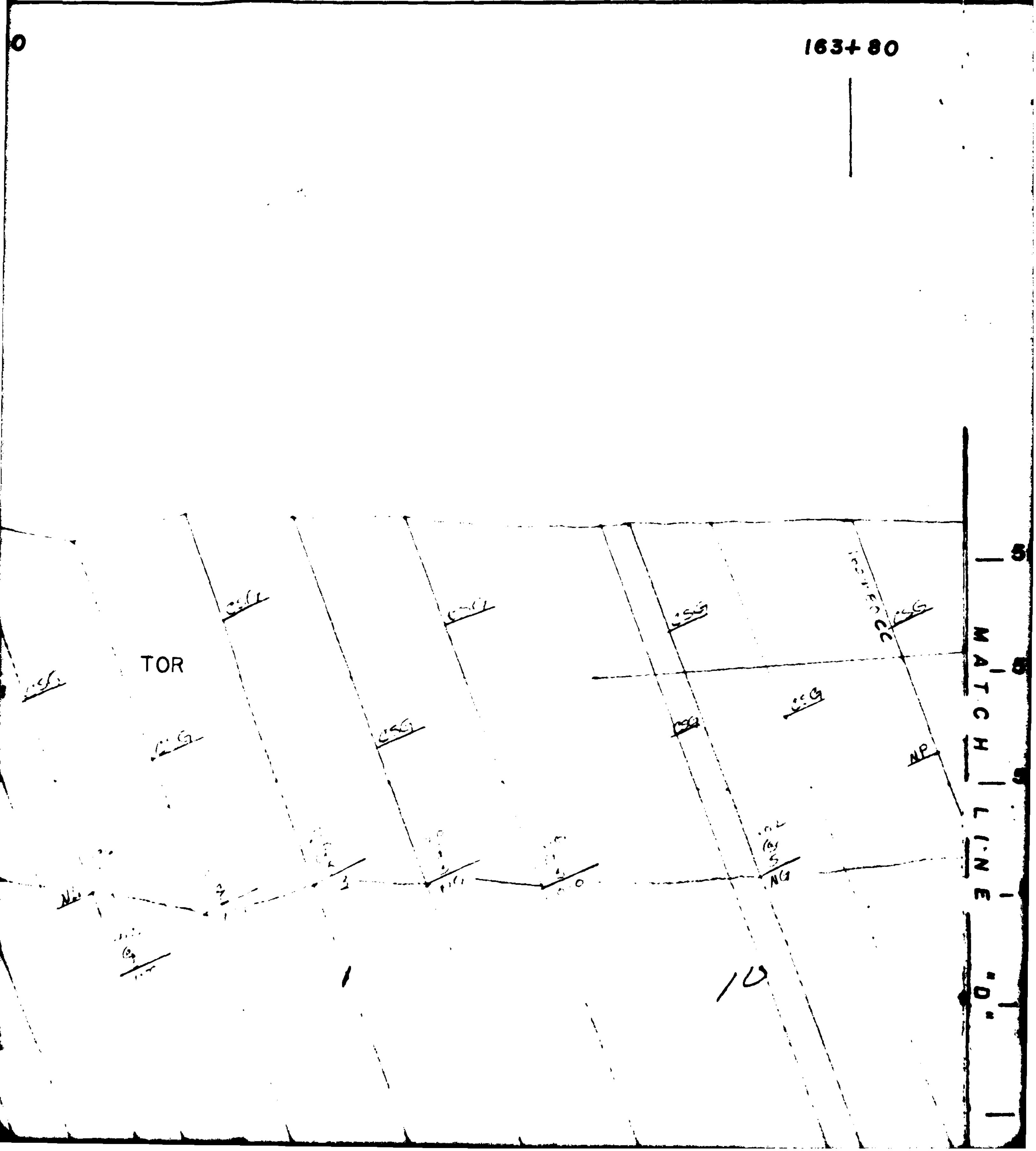
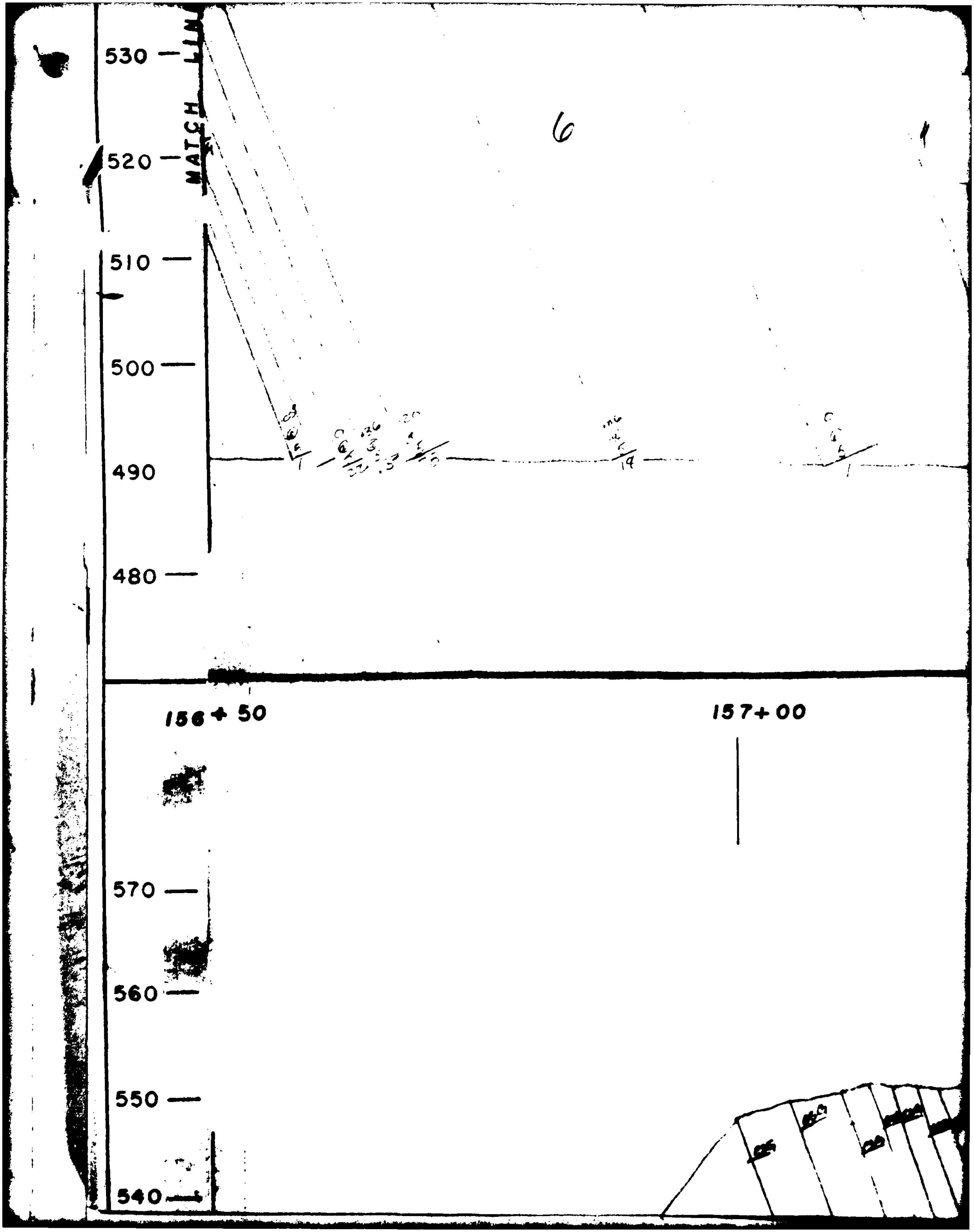
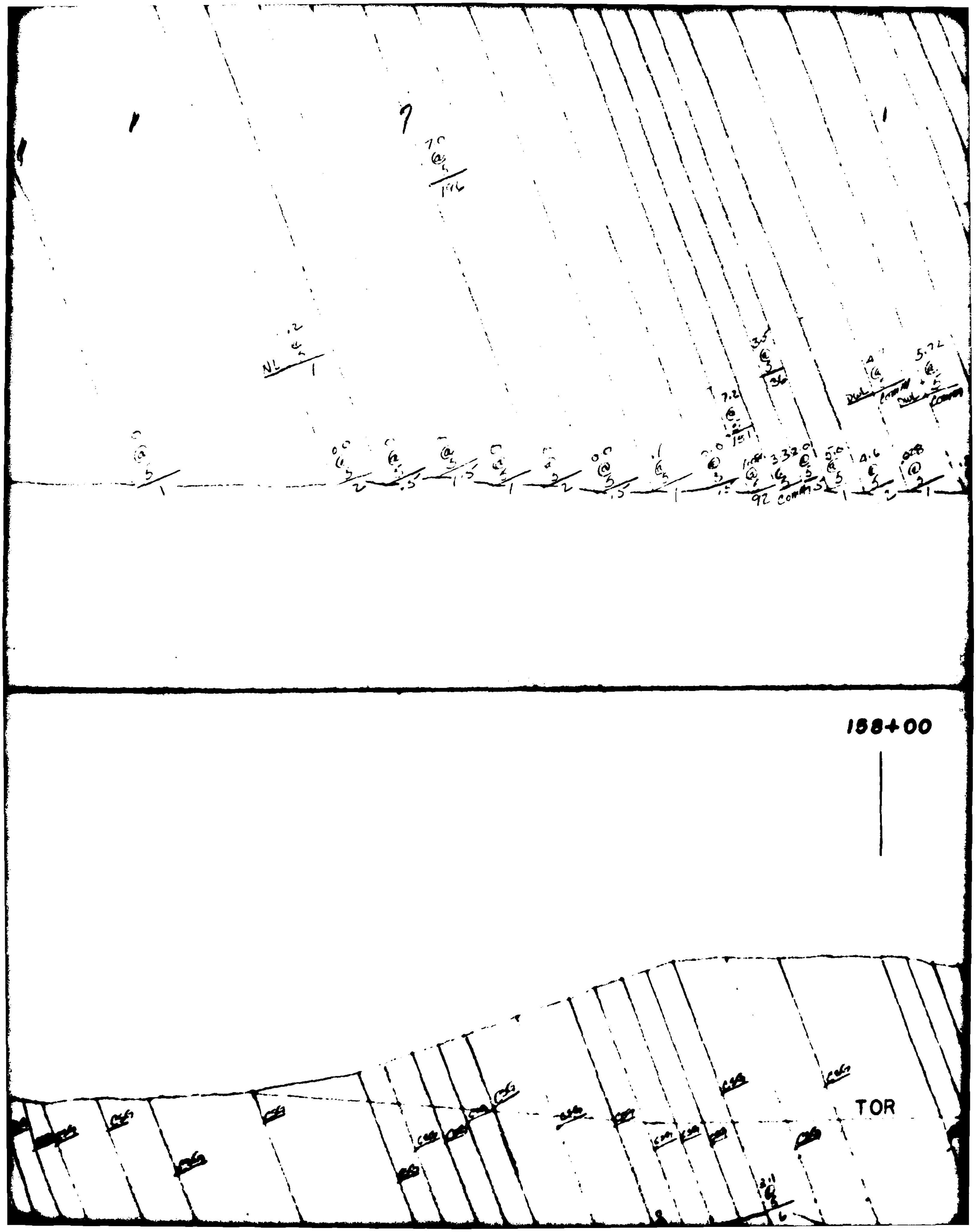
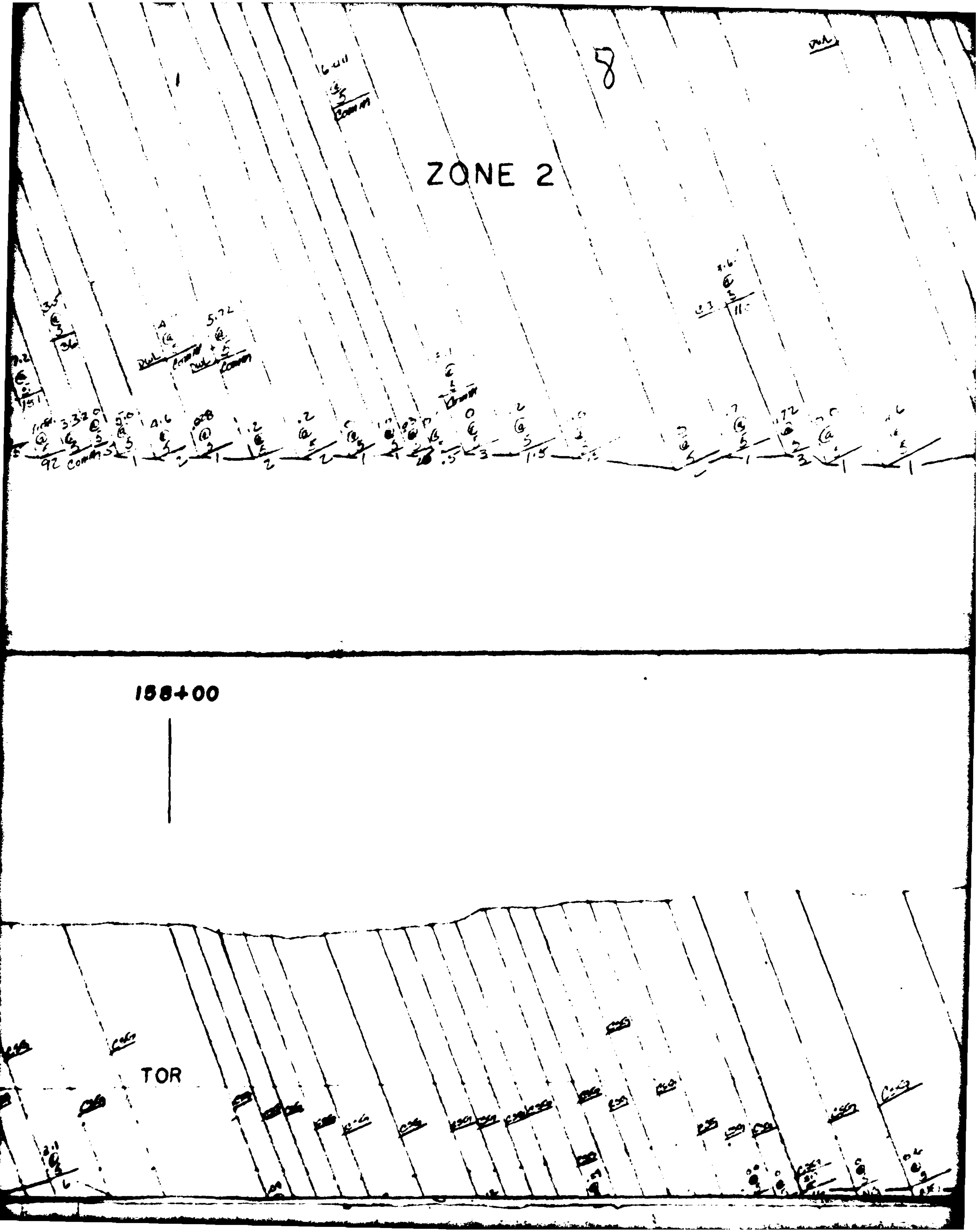
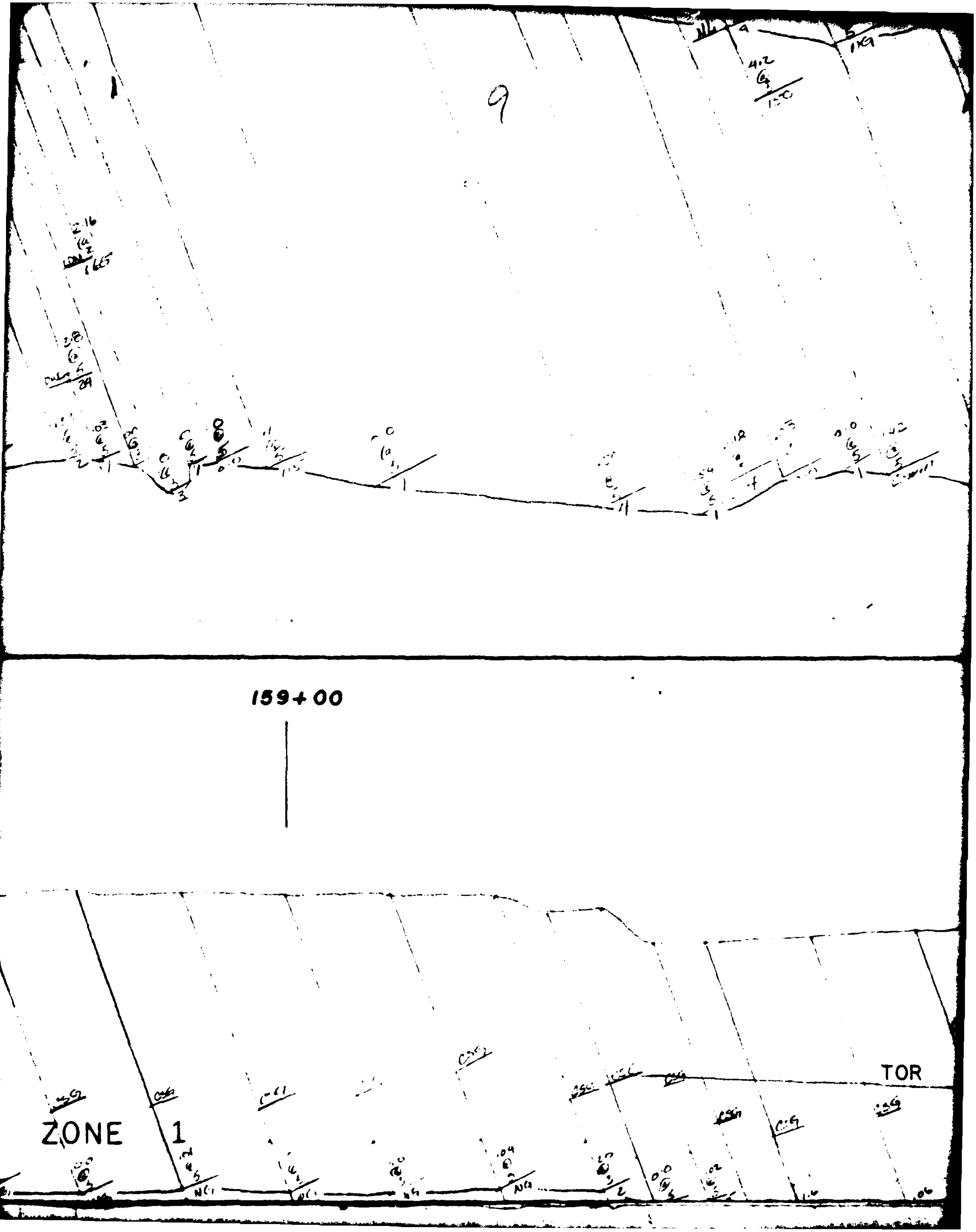


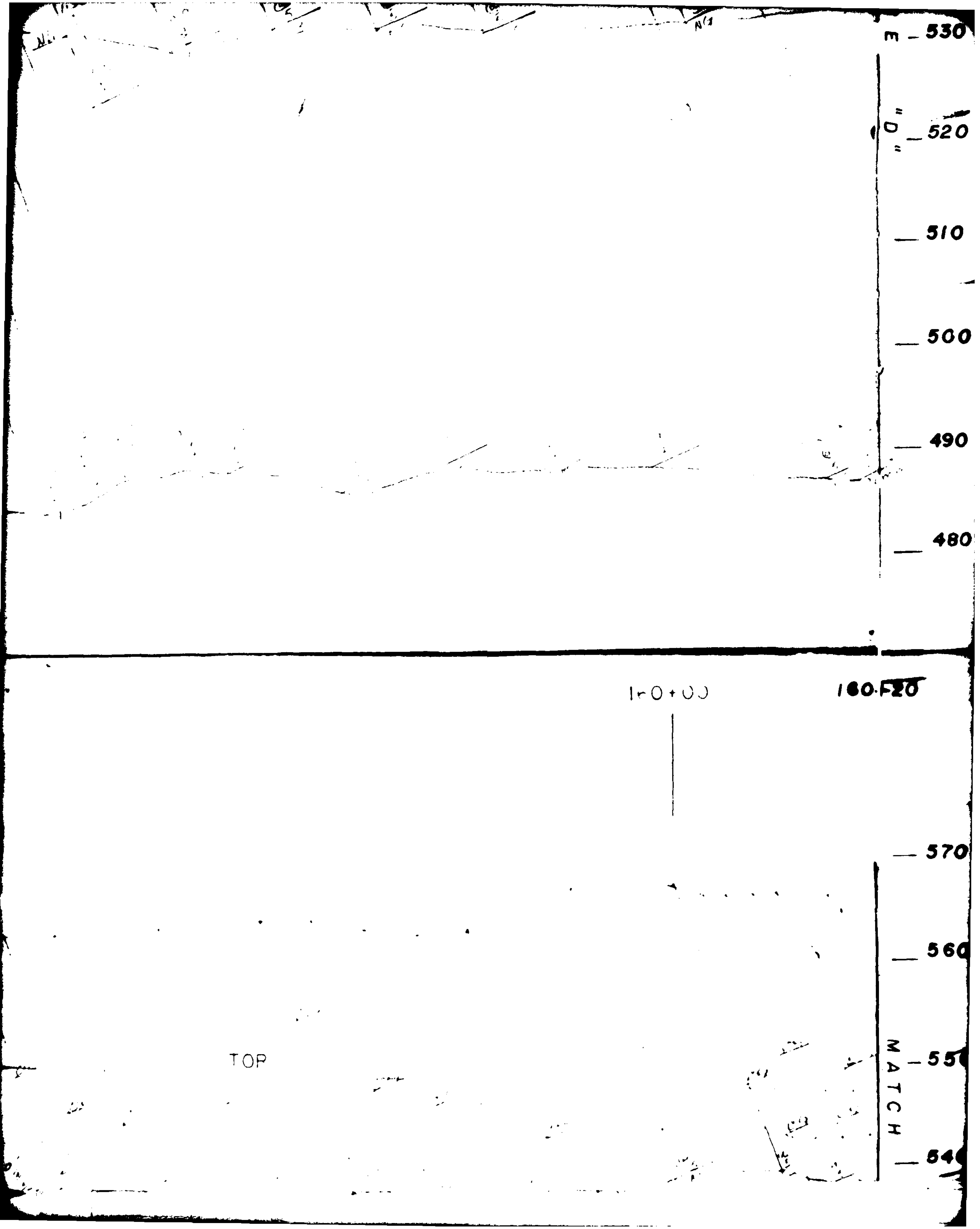

6
530
520
MATCH LIN
510
500
490
480
156+50
157+00
570
560
550
540

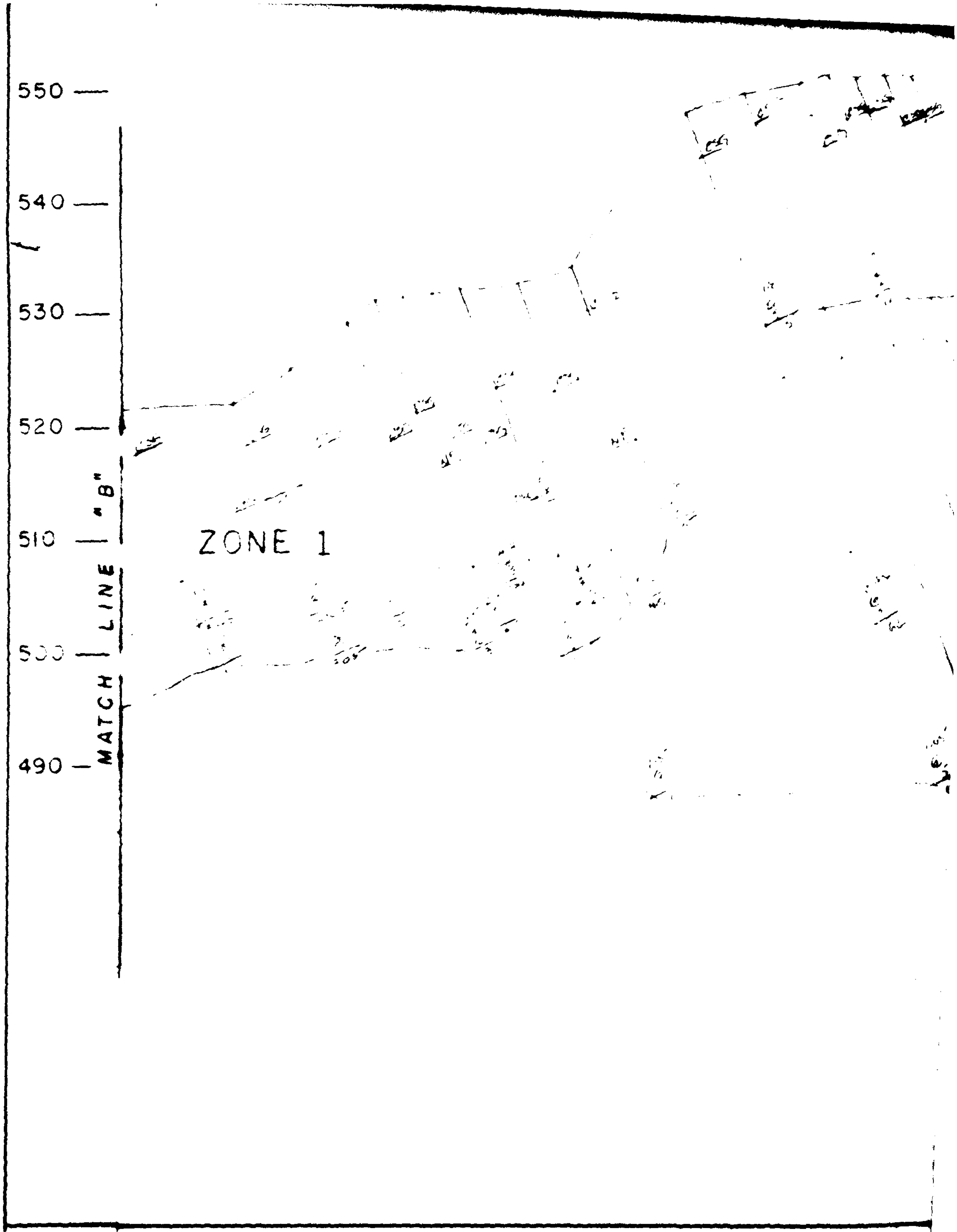


NL
158+00
TOR

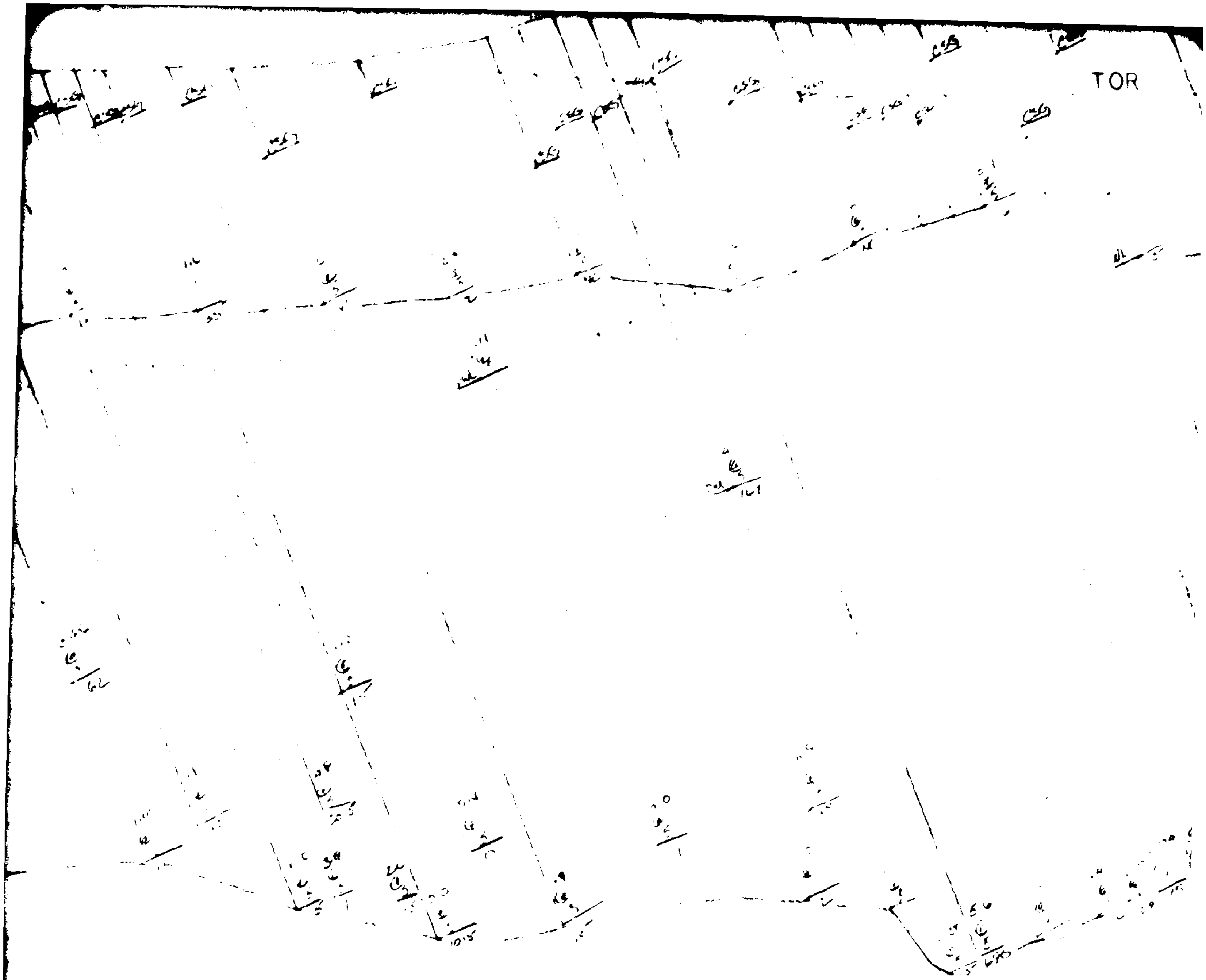

8
ZONE 2
158+00
TOR


9
159+00
TOR
ZONE 1


530
"D"
520
510
500
490
480
160·F20
570
TOP
MATCH



550
540
530
520
510
500
490
MATCH LINE "B"
ZONE 1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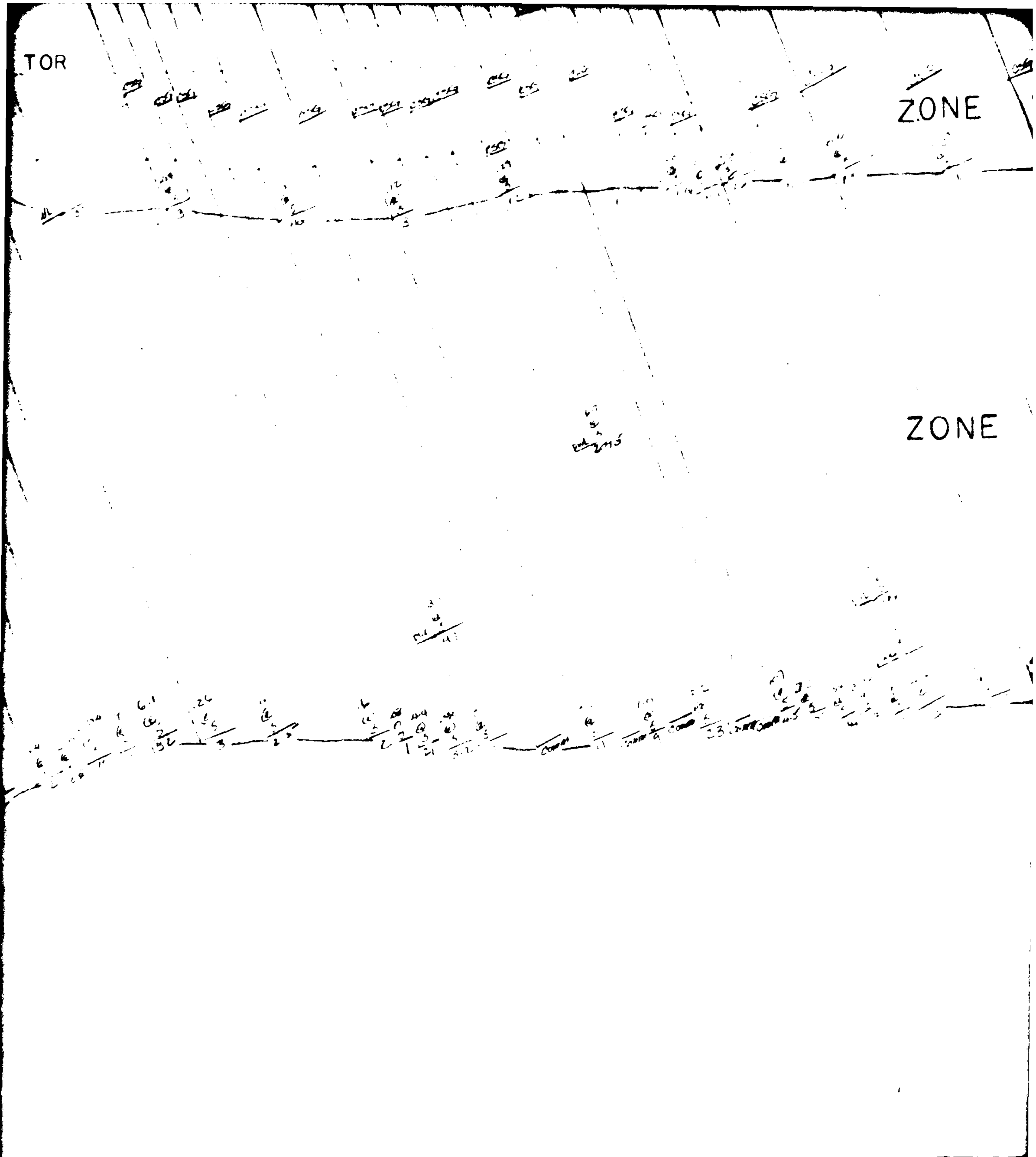
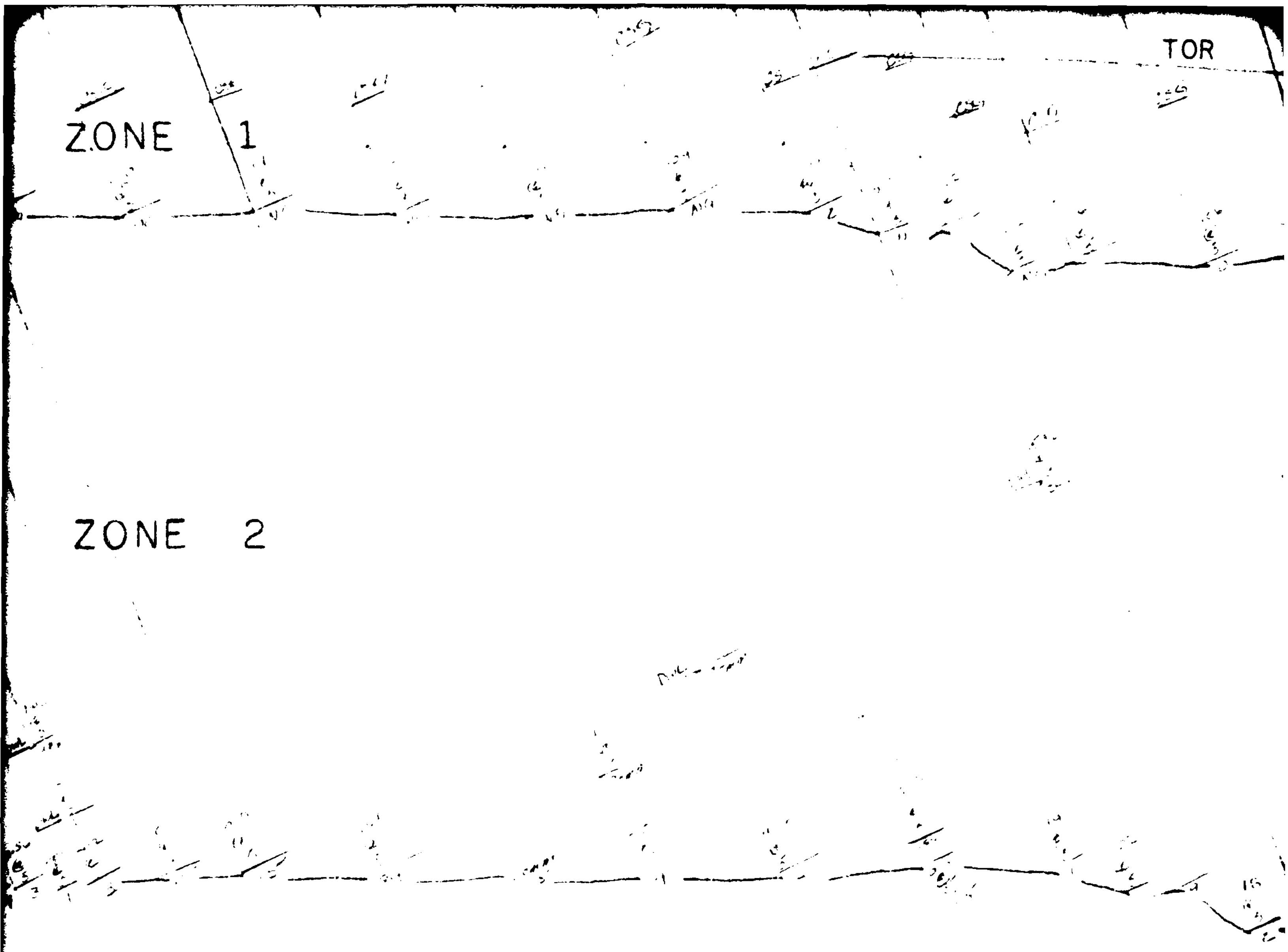
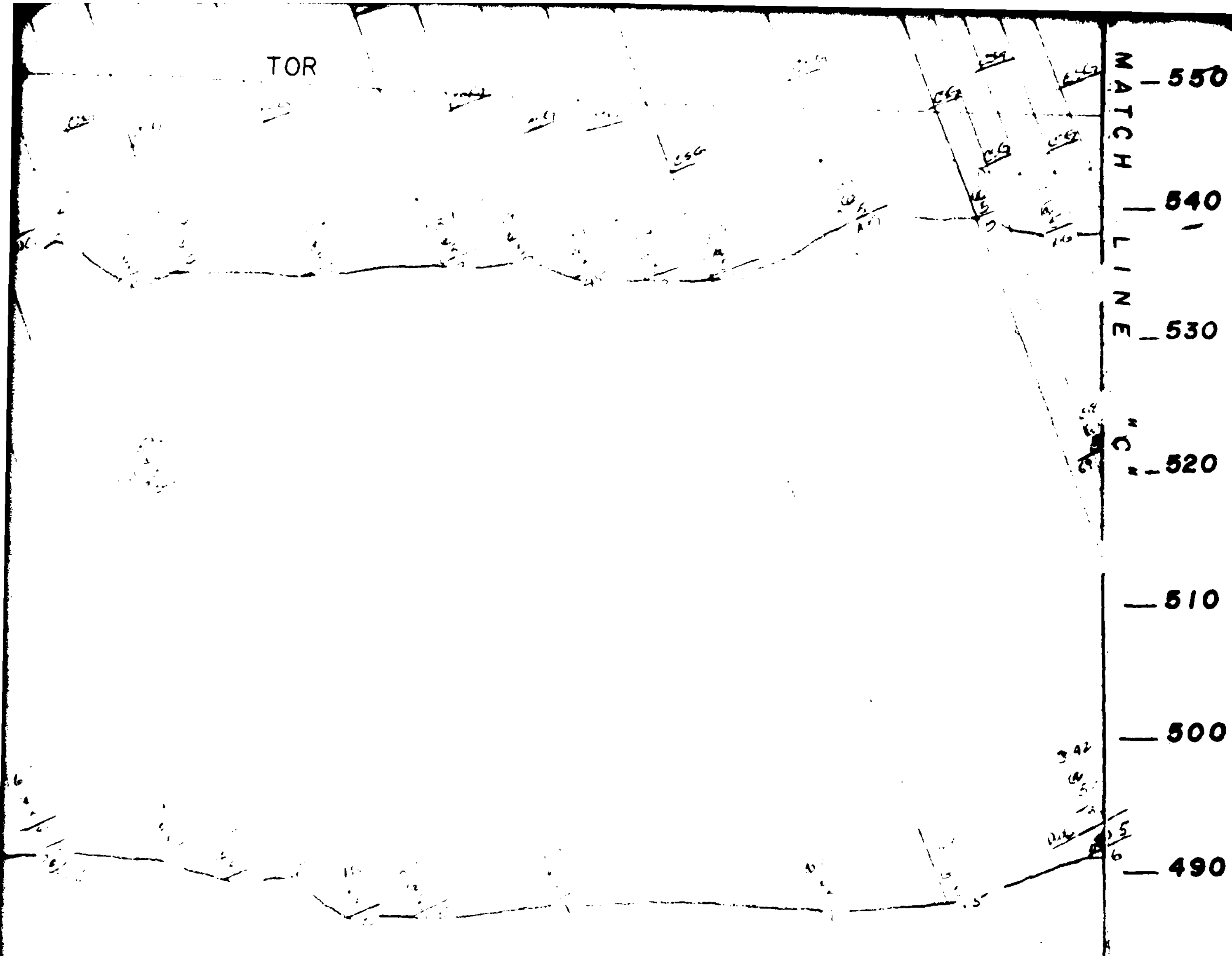

TOR
ZONE
ZONE



DESIGNED	
DRAWN:	TRACED
CHECKED:	
SUBMITTED:	
SCALE: 1" = 10'	

14



DESIGNED:	
DRAWN:	TRACED:
CHECKED:	
SUBMITTED:	

PATOKA LAKE
DAM & SPILLWAY
GROUT PROFILE
DOWN STREAM LINE

STATION 156+50 to 160+20 STATION 160+20 to 163+90 CC

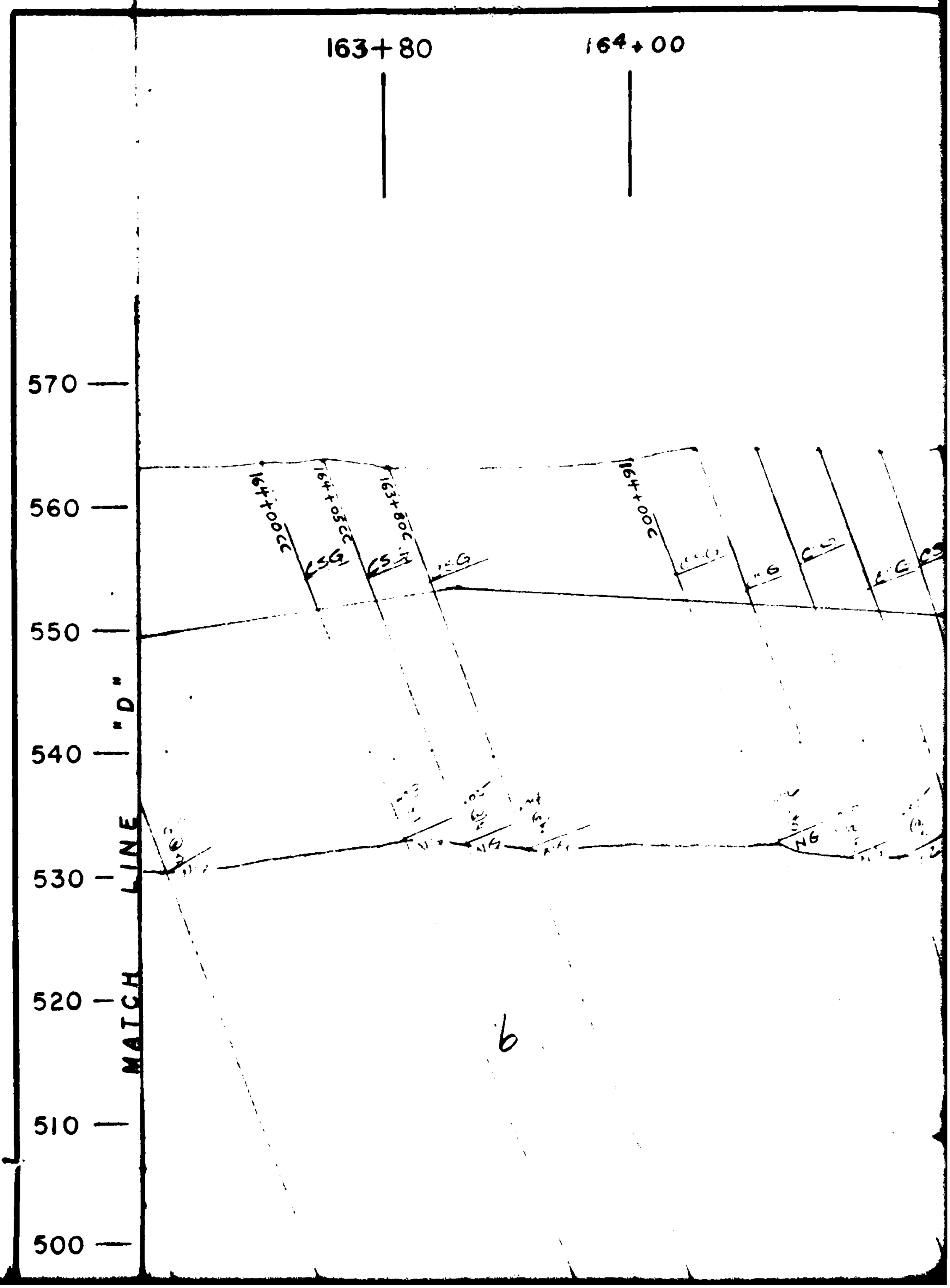
SCALE: 1" = 10' NATURAL

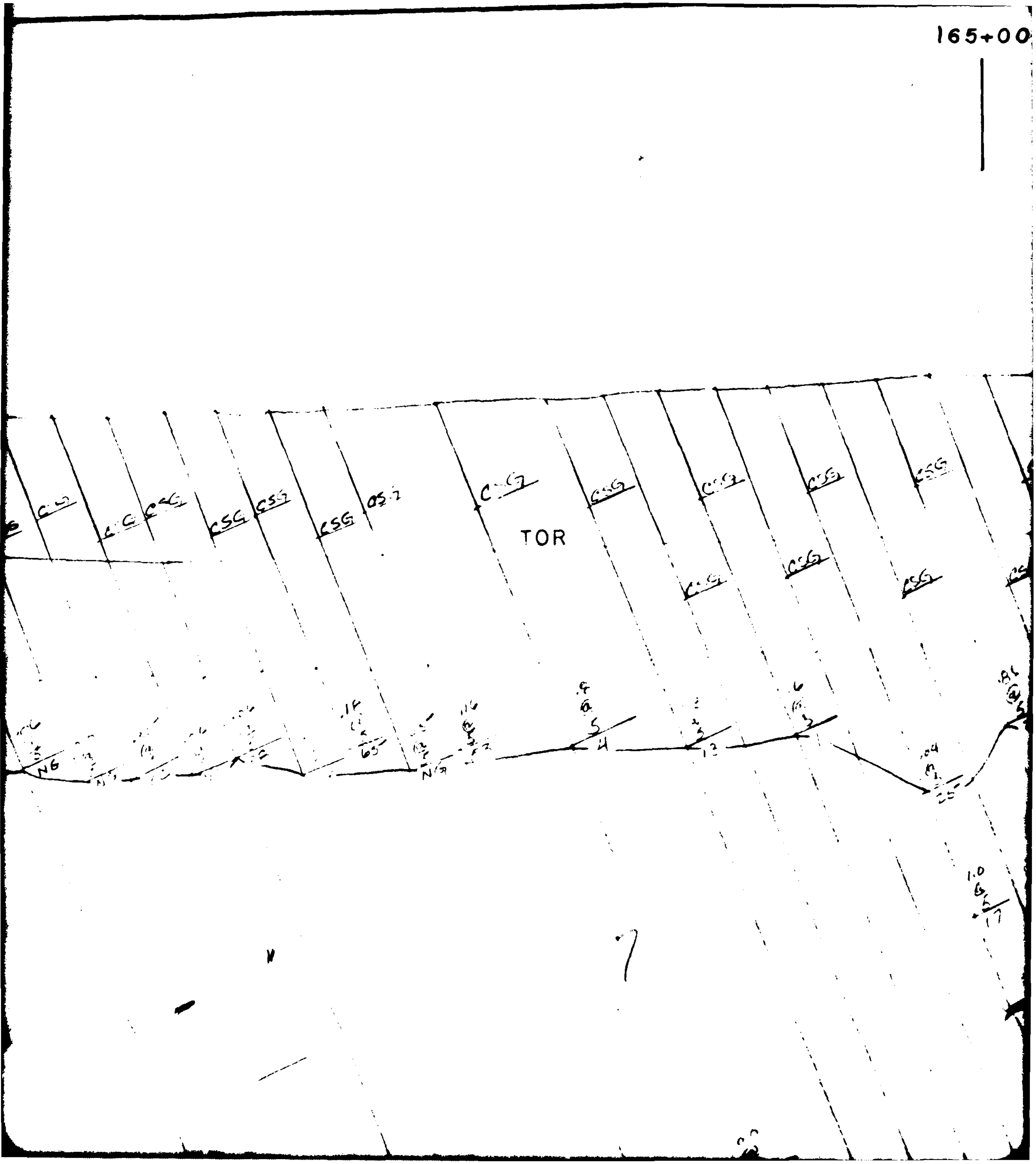
DATE:

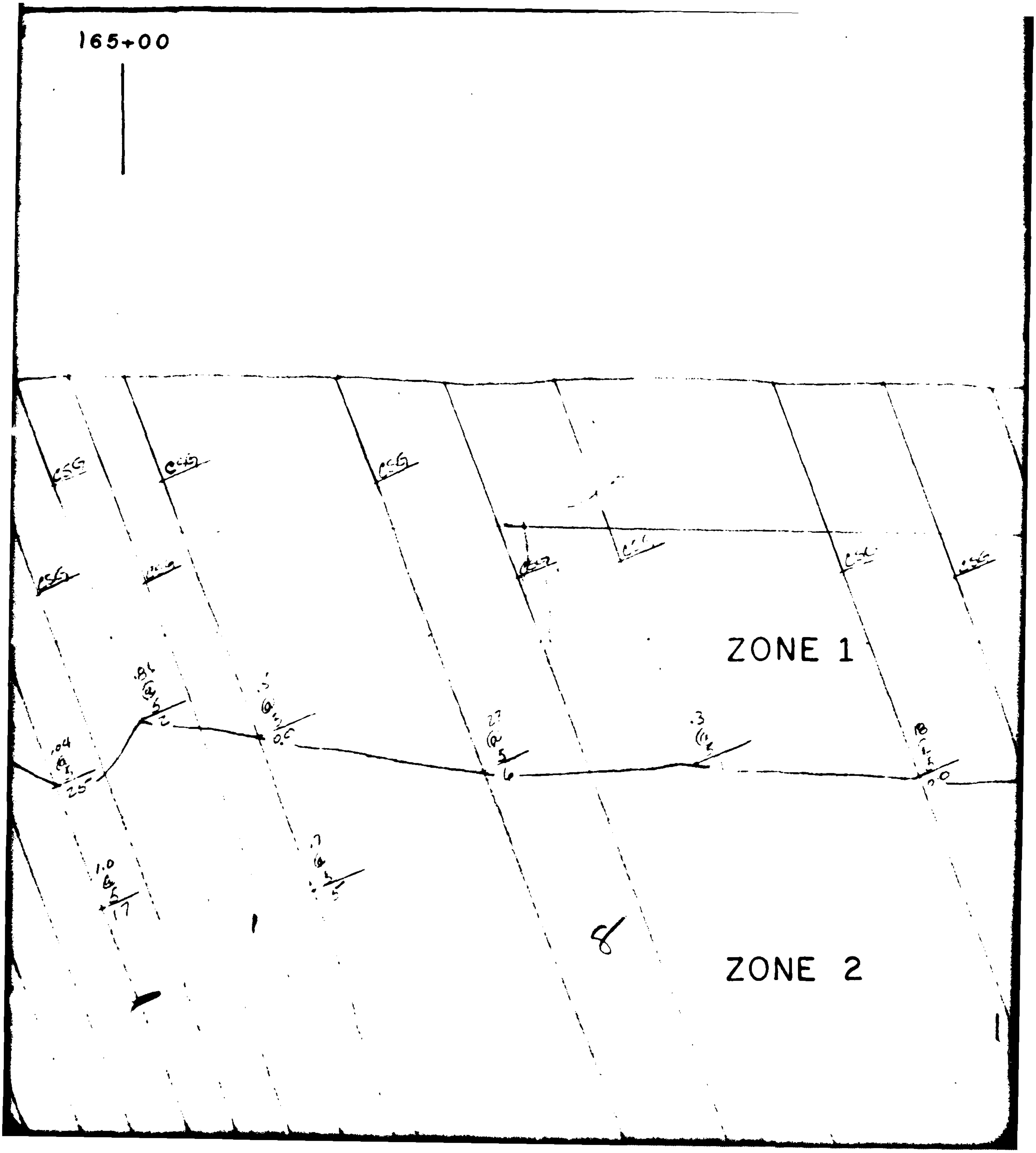
DRAWING NUMBER
FIGURE C-3

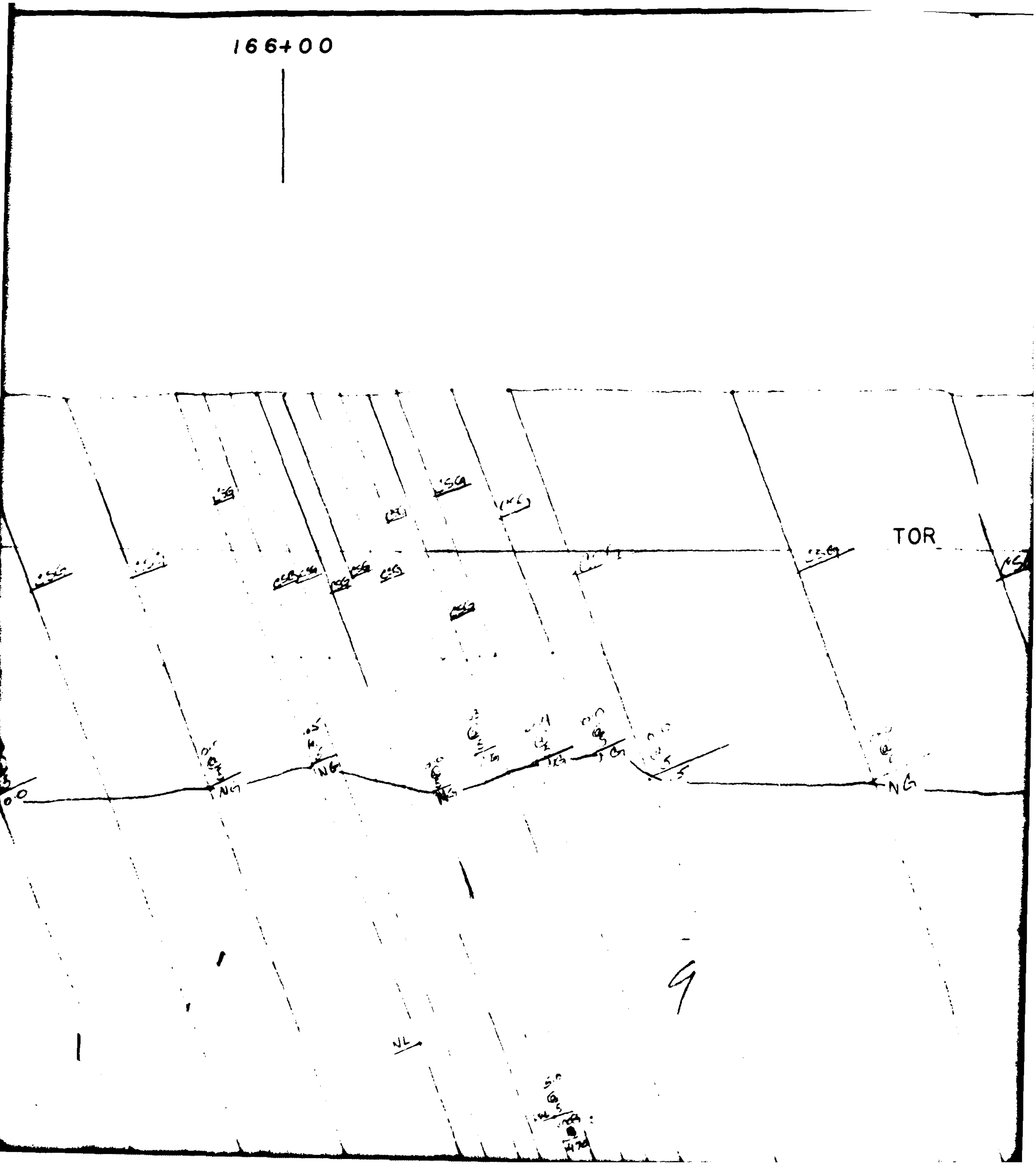
1 15

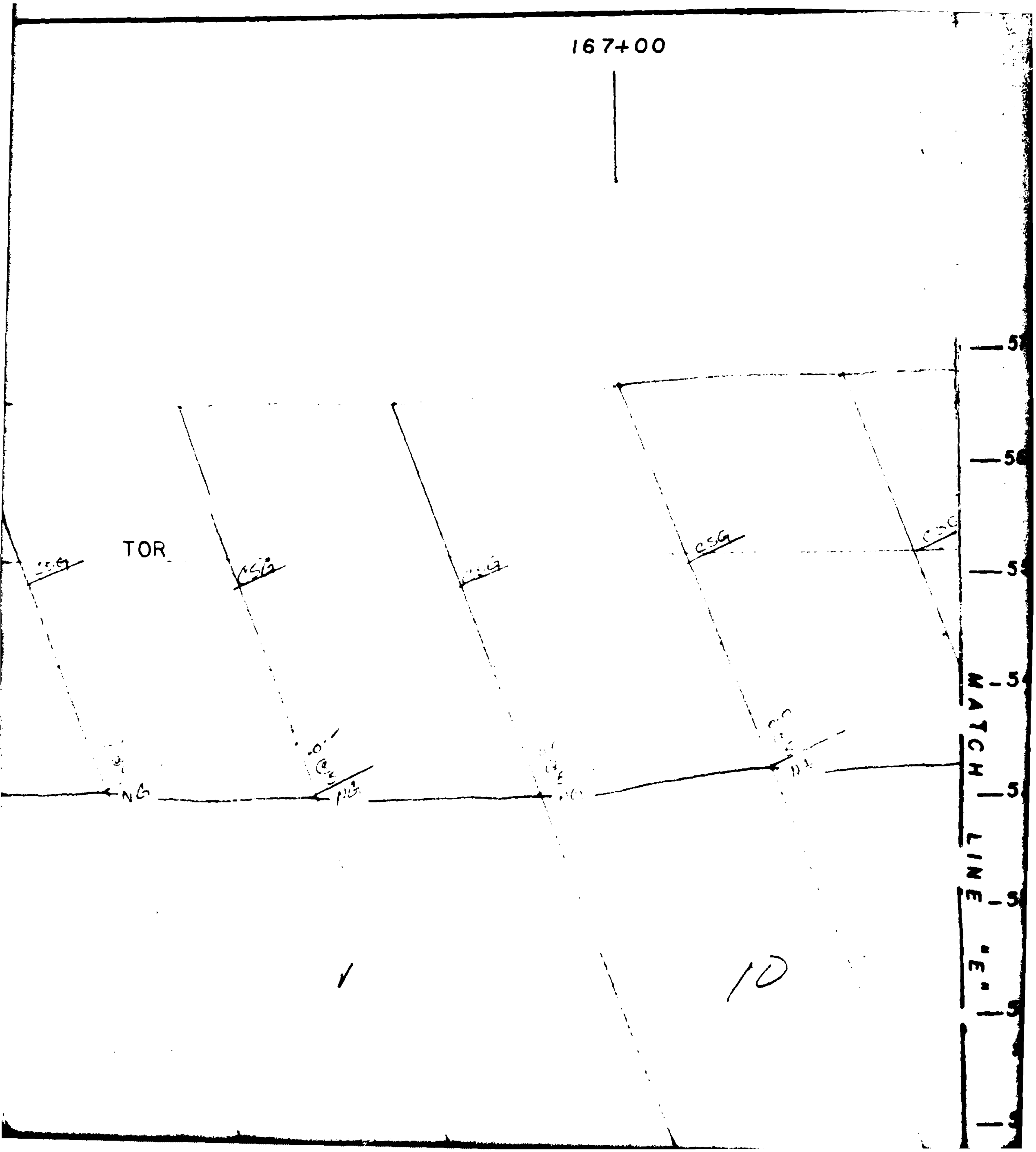
1


163+80
164+00
570
560
550
540
530
520
510
500
MATCH LINE "D"
164+00CL
164+03CL
163+80CL
164+00C
CSG
NG
6

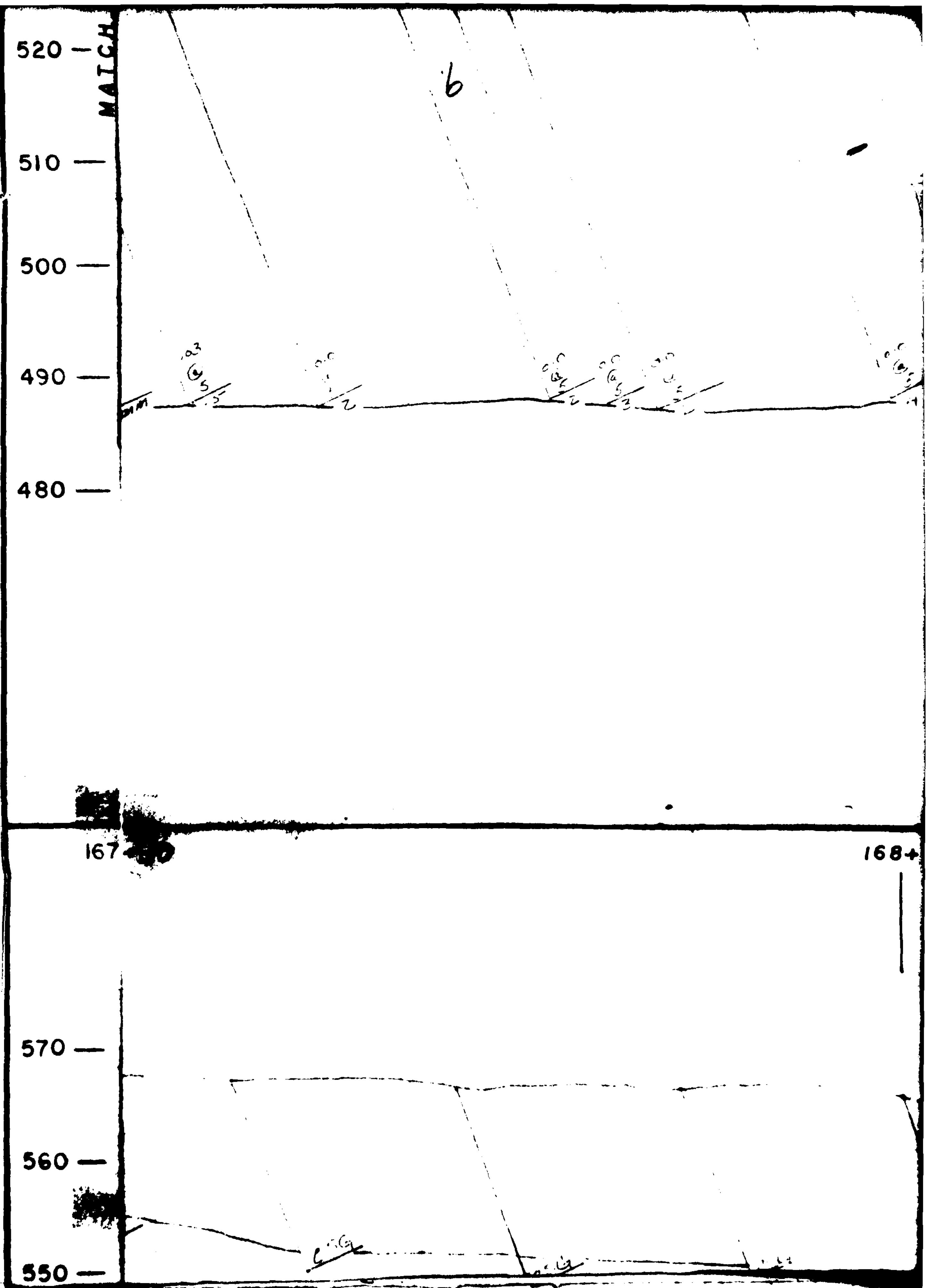

165+00
TOR
CSG
NG

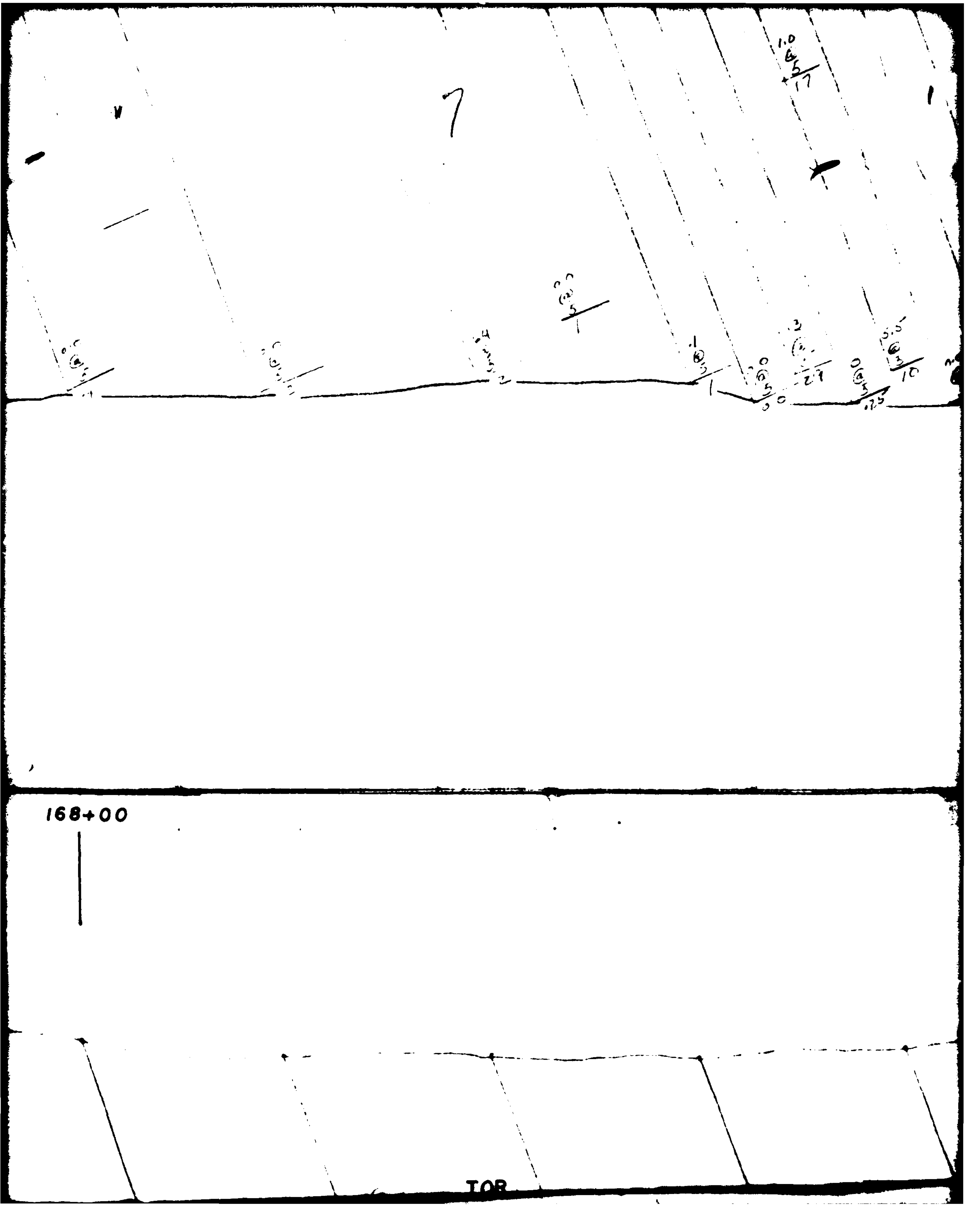

165+00
CSG
ZONE 1
ZONE 2


166+00
TOR
NG
NL
9

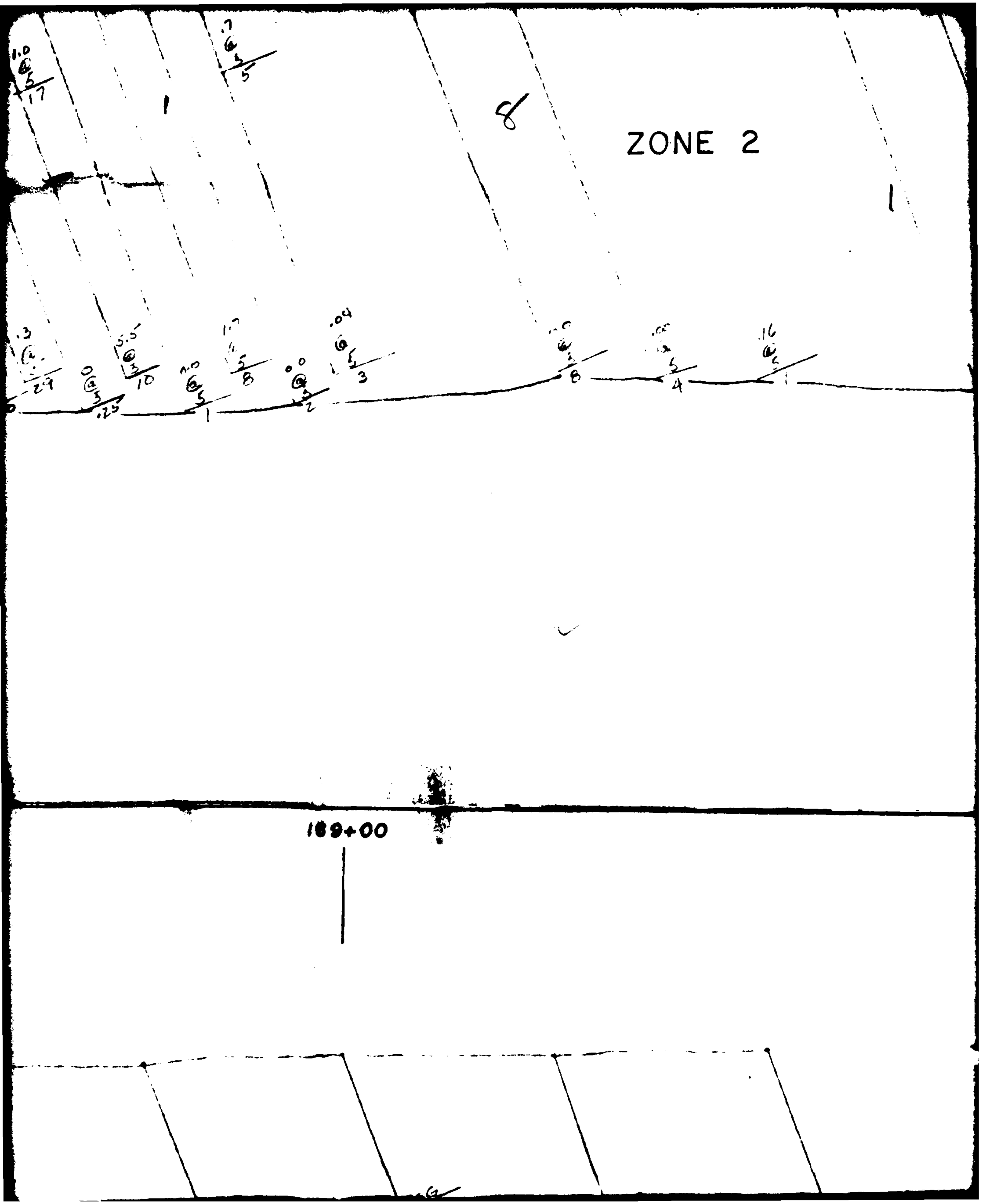


167+00
TOR
CSG
CSG
CSG
NG
MATCH LINE "E"
1
10


6
MATCH
520
510
500
490
480
167+
168+
570
560
550

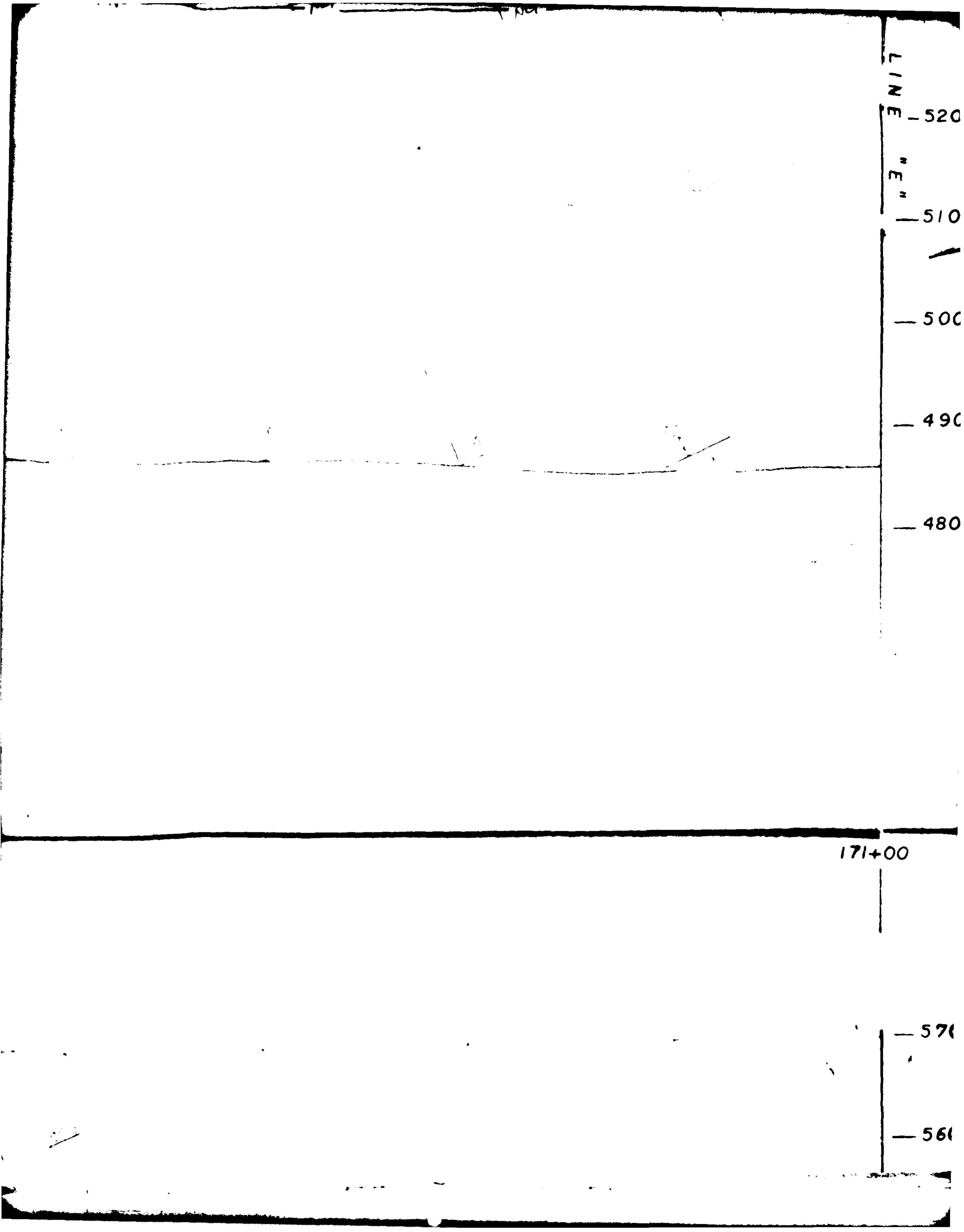


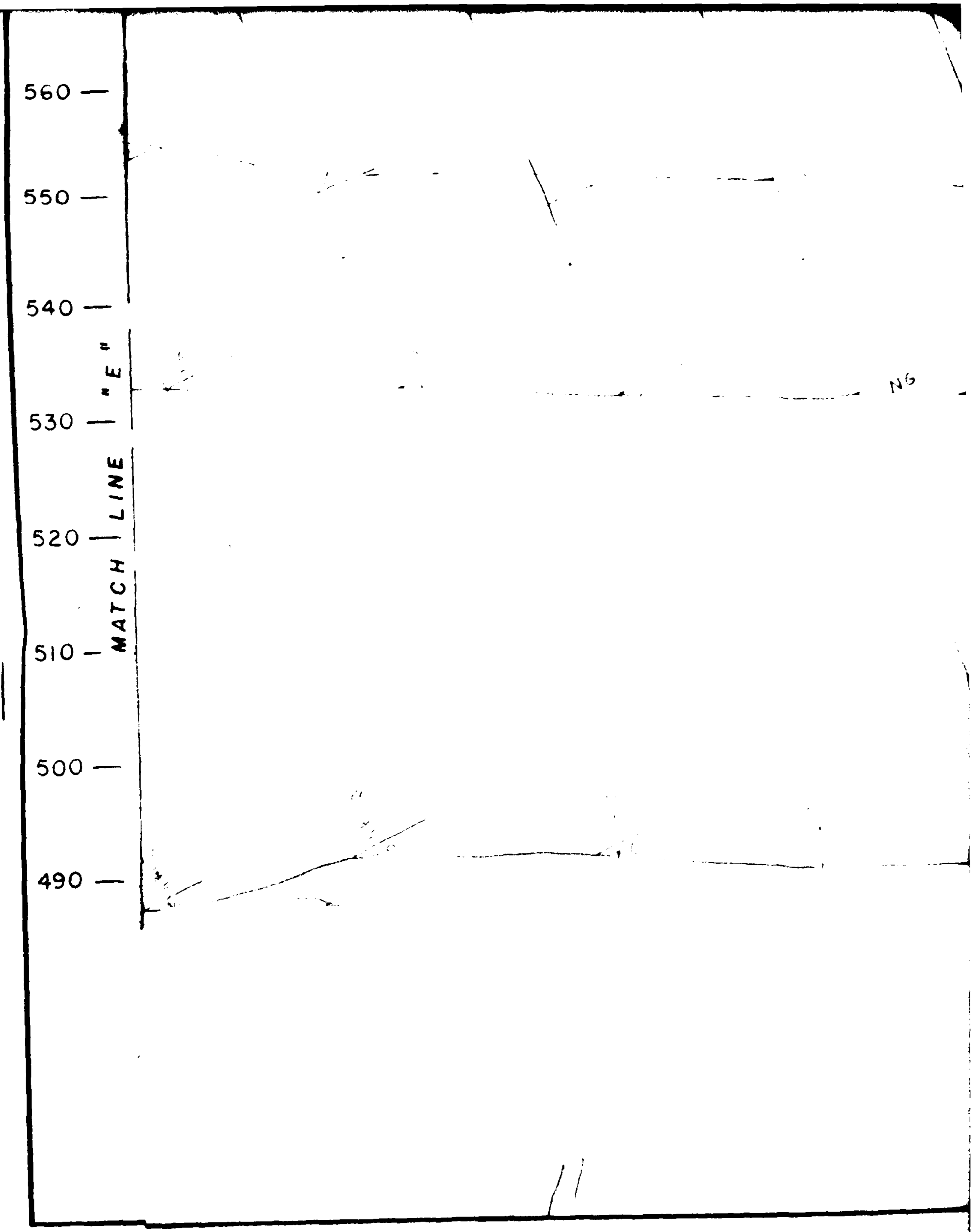
7
168+00
TOR


ZONE 2
189+00
8

9

170+00

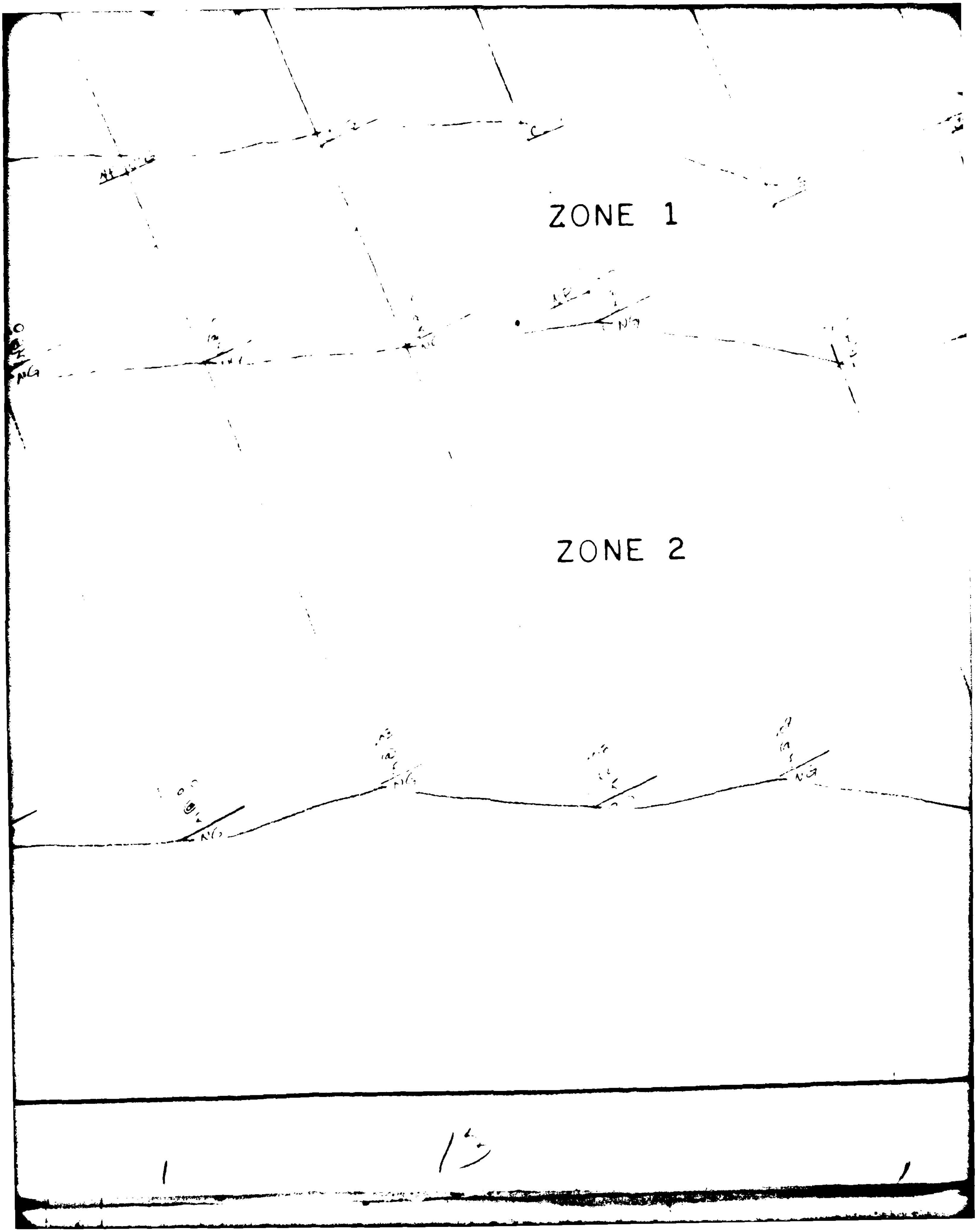

LINE "E"
510
480
171+00

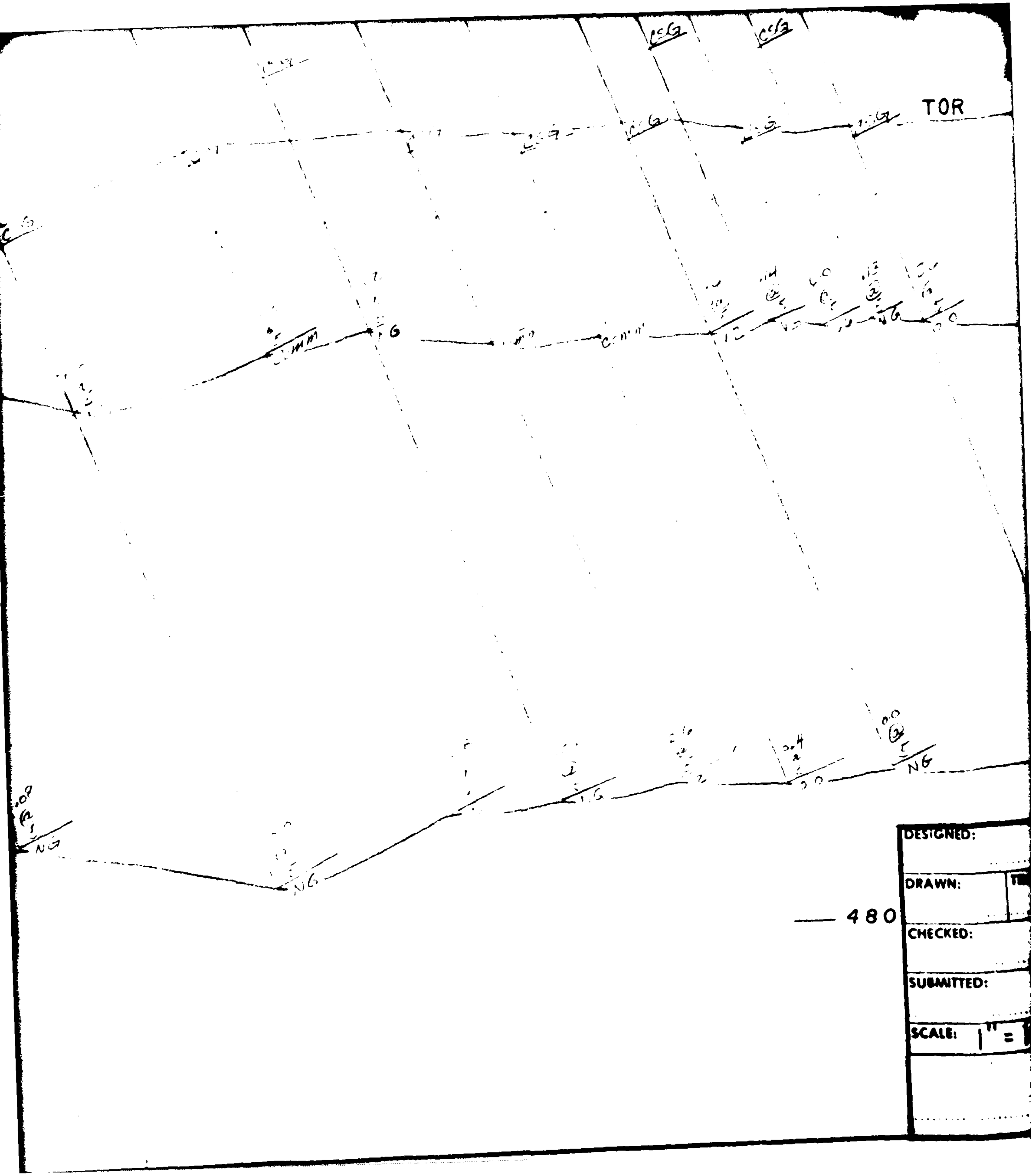

560
550
540
530
520
510
500
490
MATCH LINE "E"
NG

TOR

NG

12


ZONE 1
ZONE 2



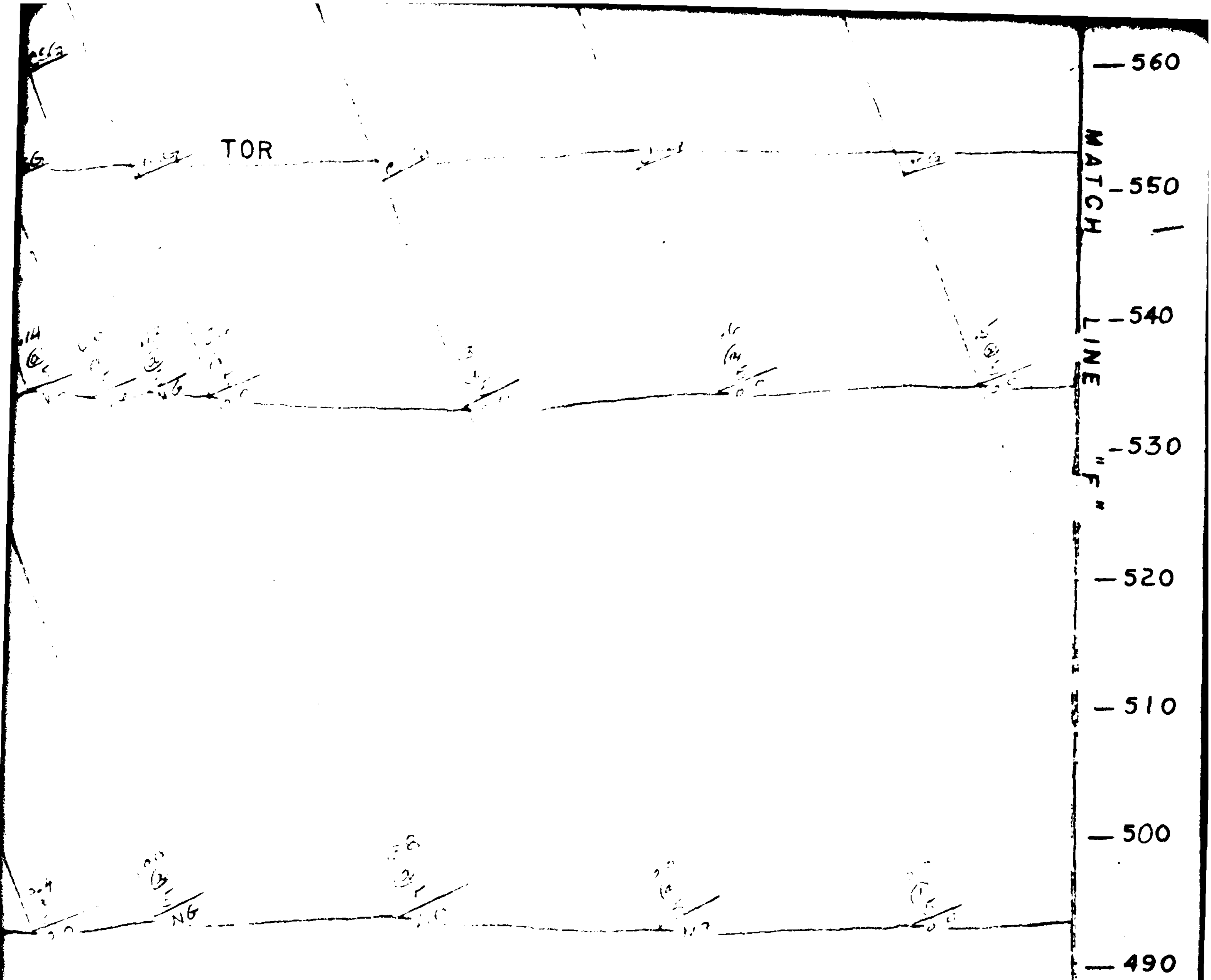
DESIGNED:

DRAWN:

CHECKED:

SUBMITTED:

SCALE: 1" =



DESIGNED:

DRAWN: TRACED:

CHECKED:

SUBMITTED:

PATOKA LAKE
DAM & SPILLWAY
GROUT PROFILE
DOWN STREAM LINE

STATION 163+90CC to 167+30 STATION 167+30 to 171+00

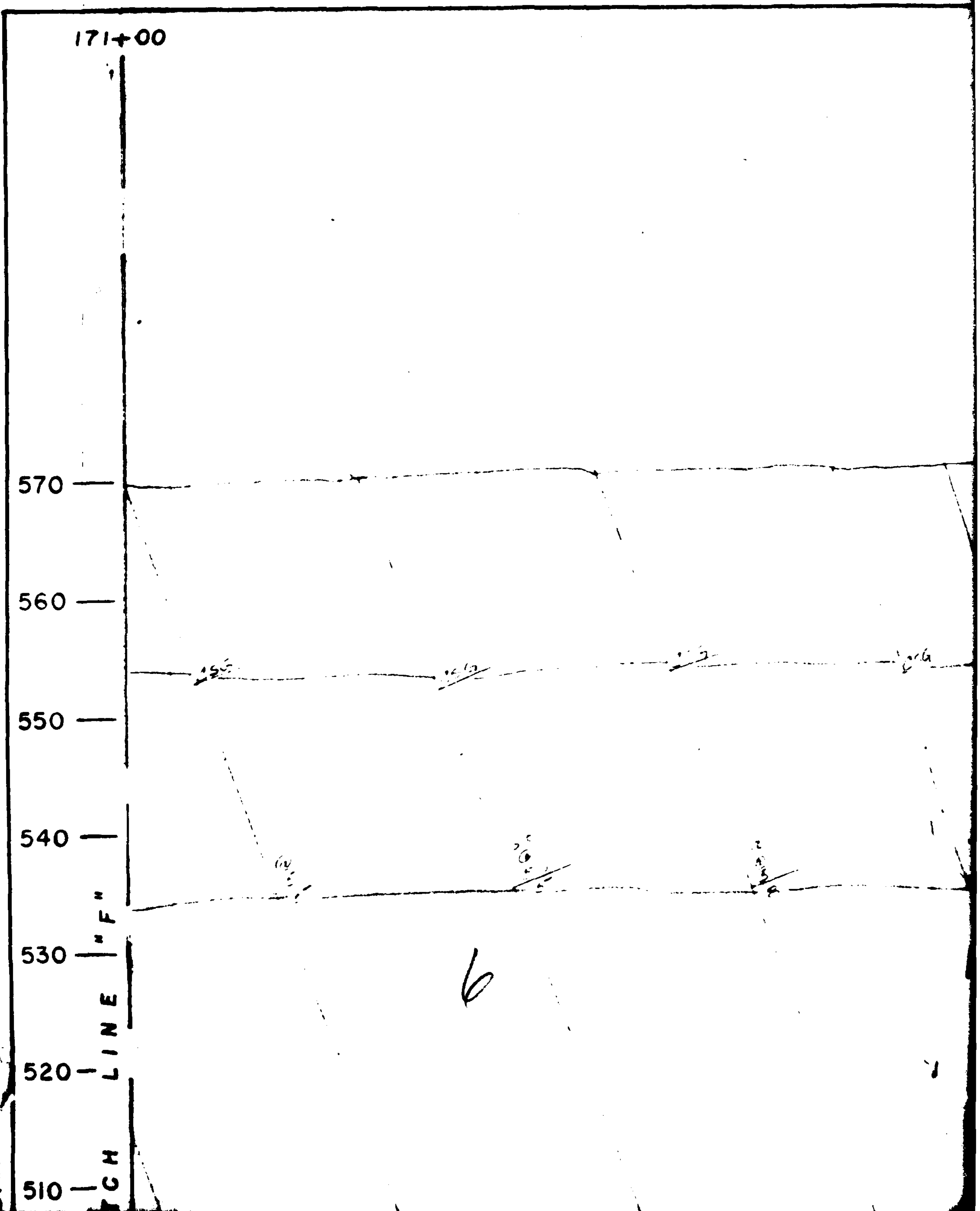
SCALE: 1" = 10' NATURAL

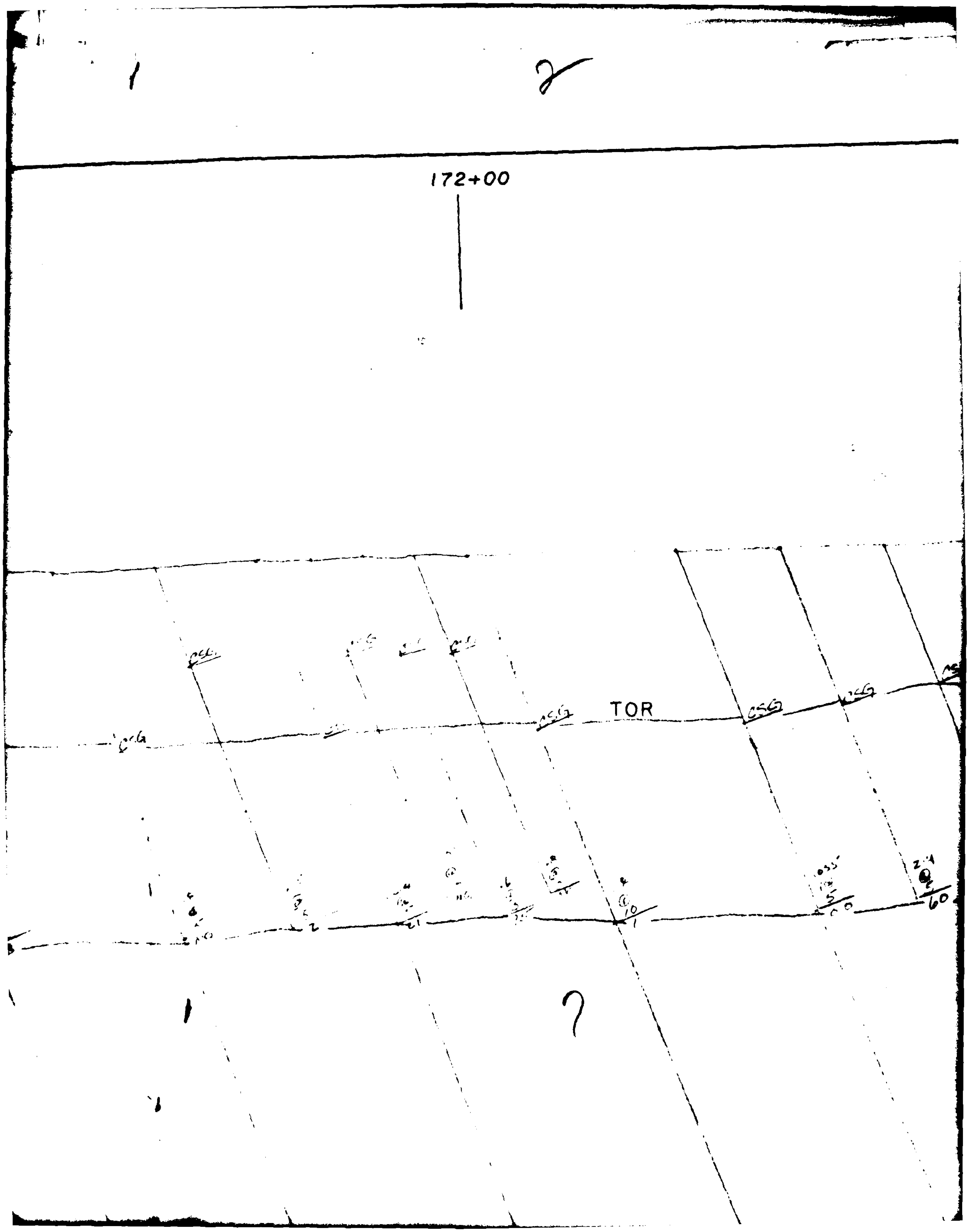
DATE:

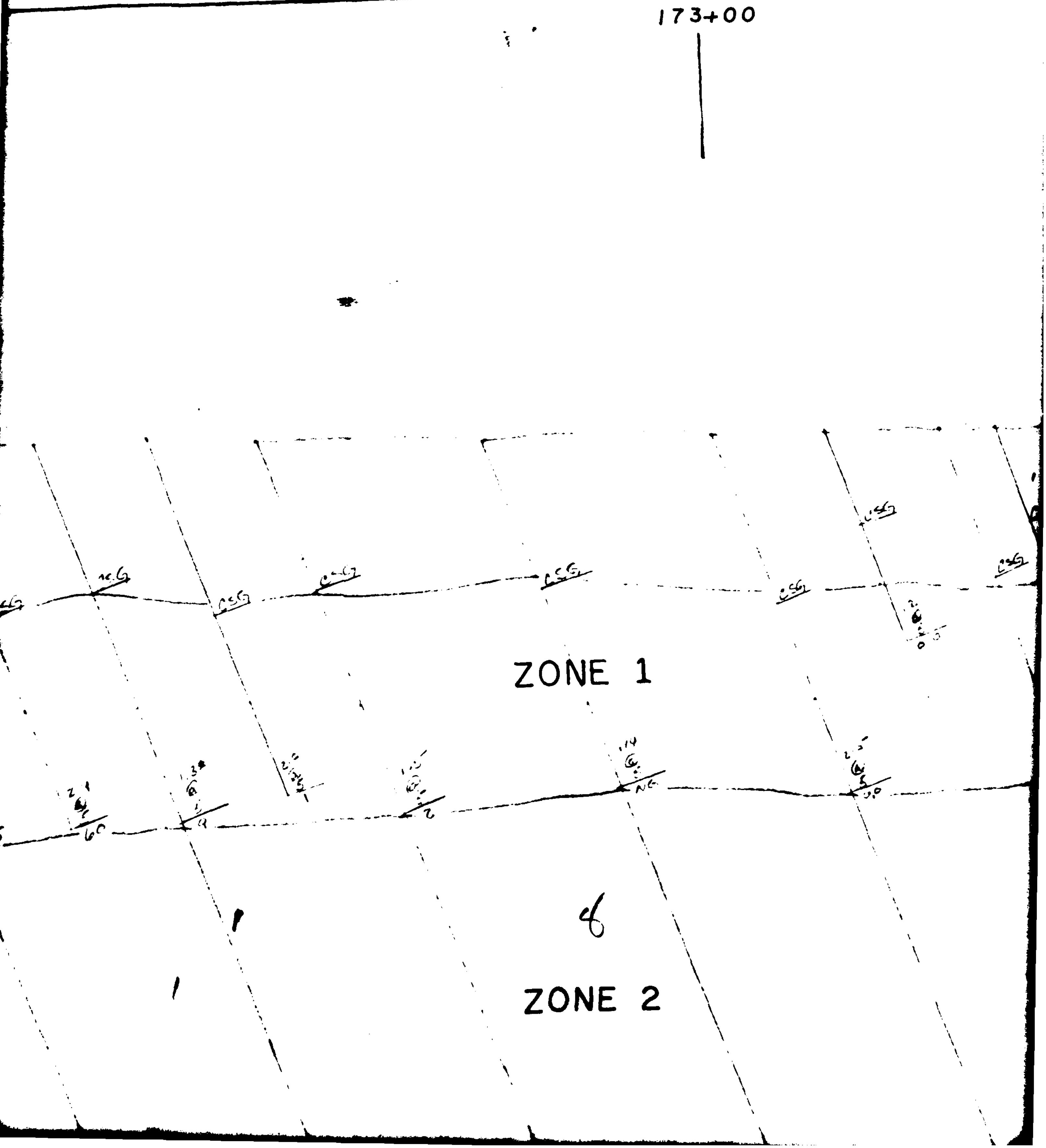
DRAWING NUMBER
FIGURE C-4

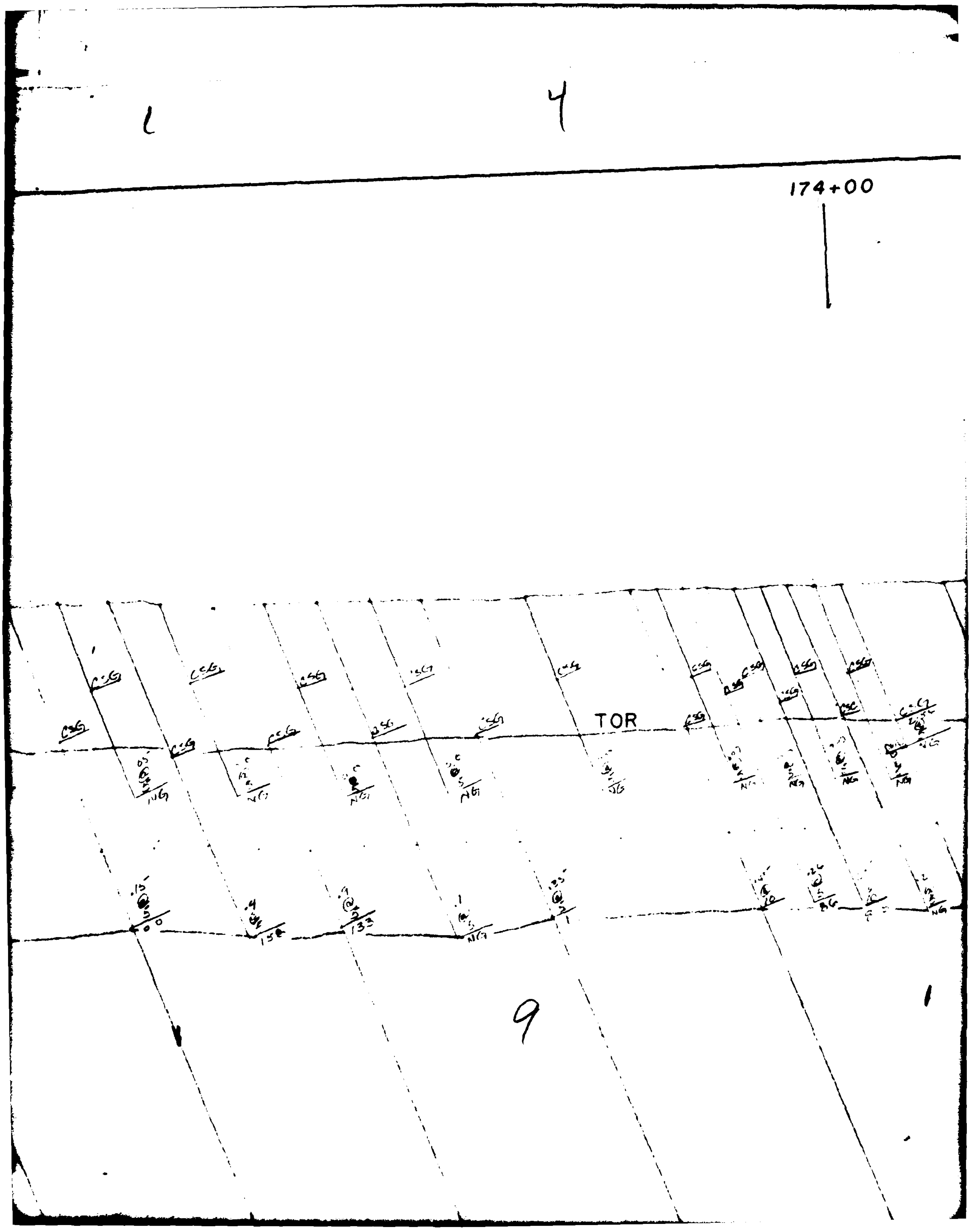
15

1

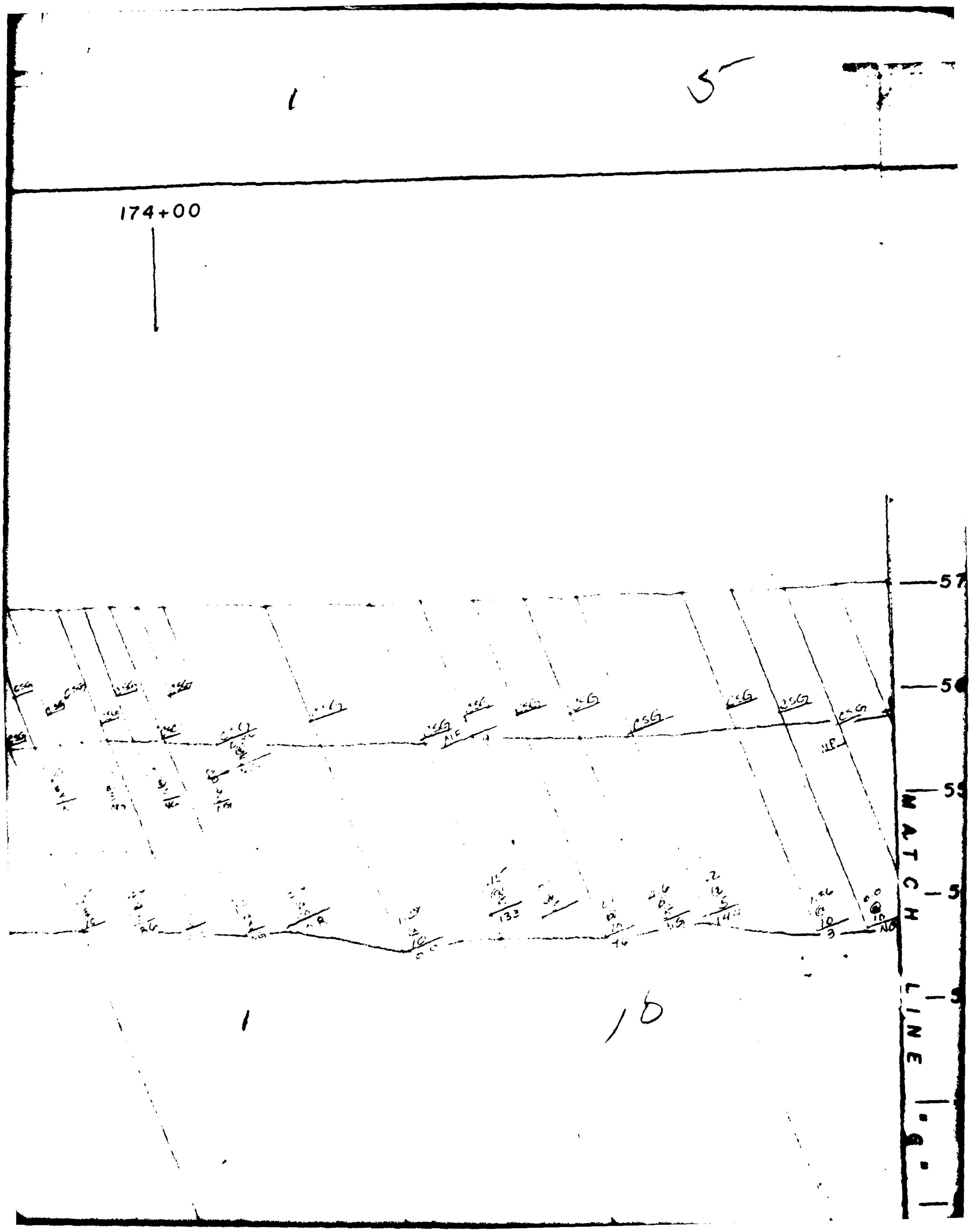



172+00
TOR

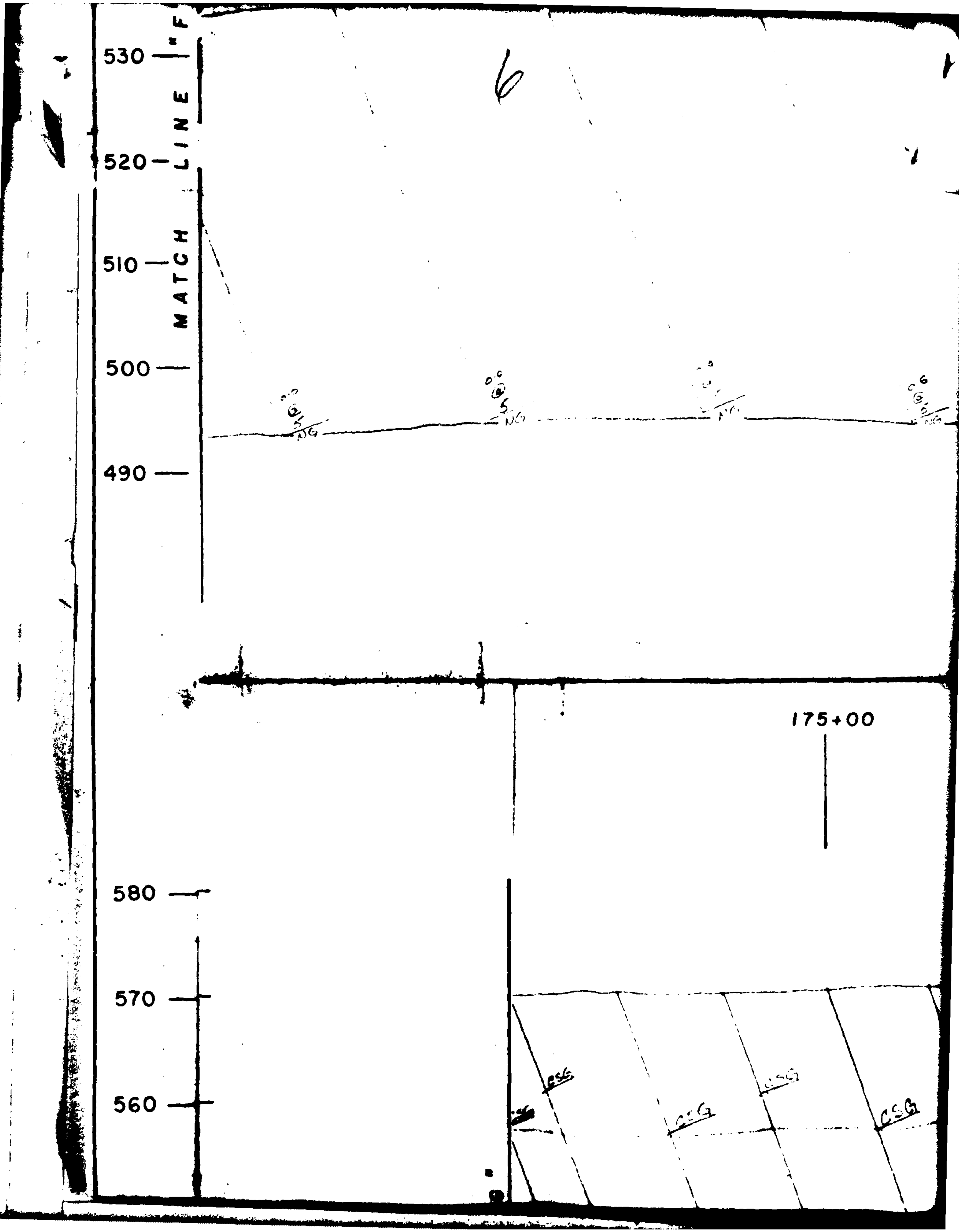

173+00
ZONE 1
ZONE 2

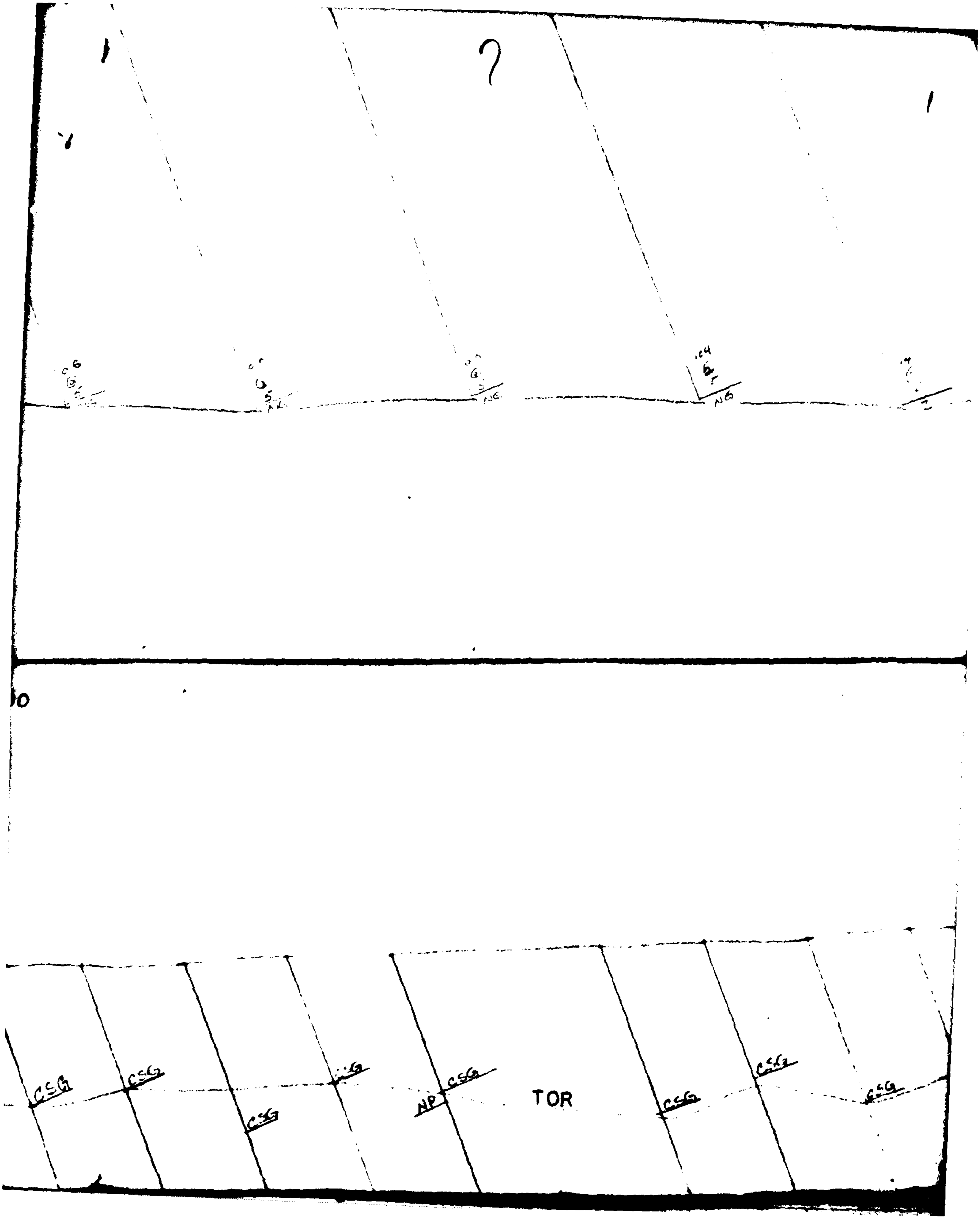


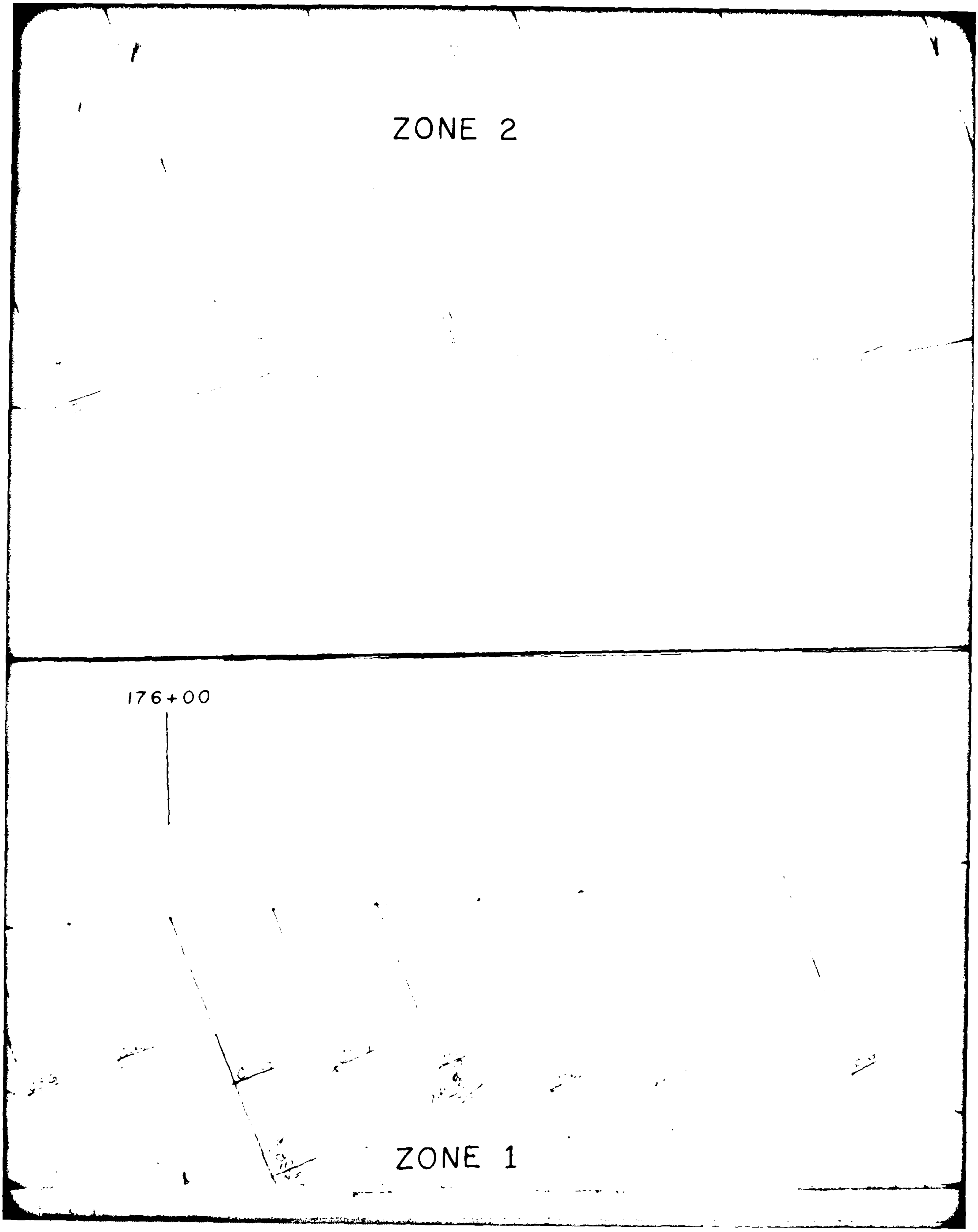
4
174+00
TOR
9

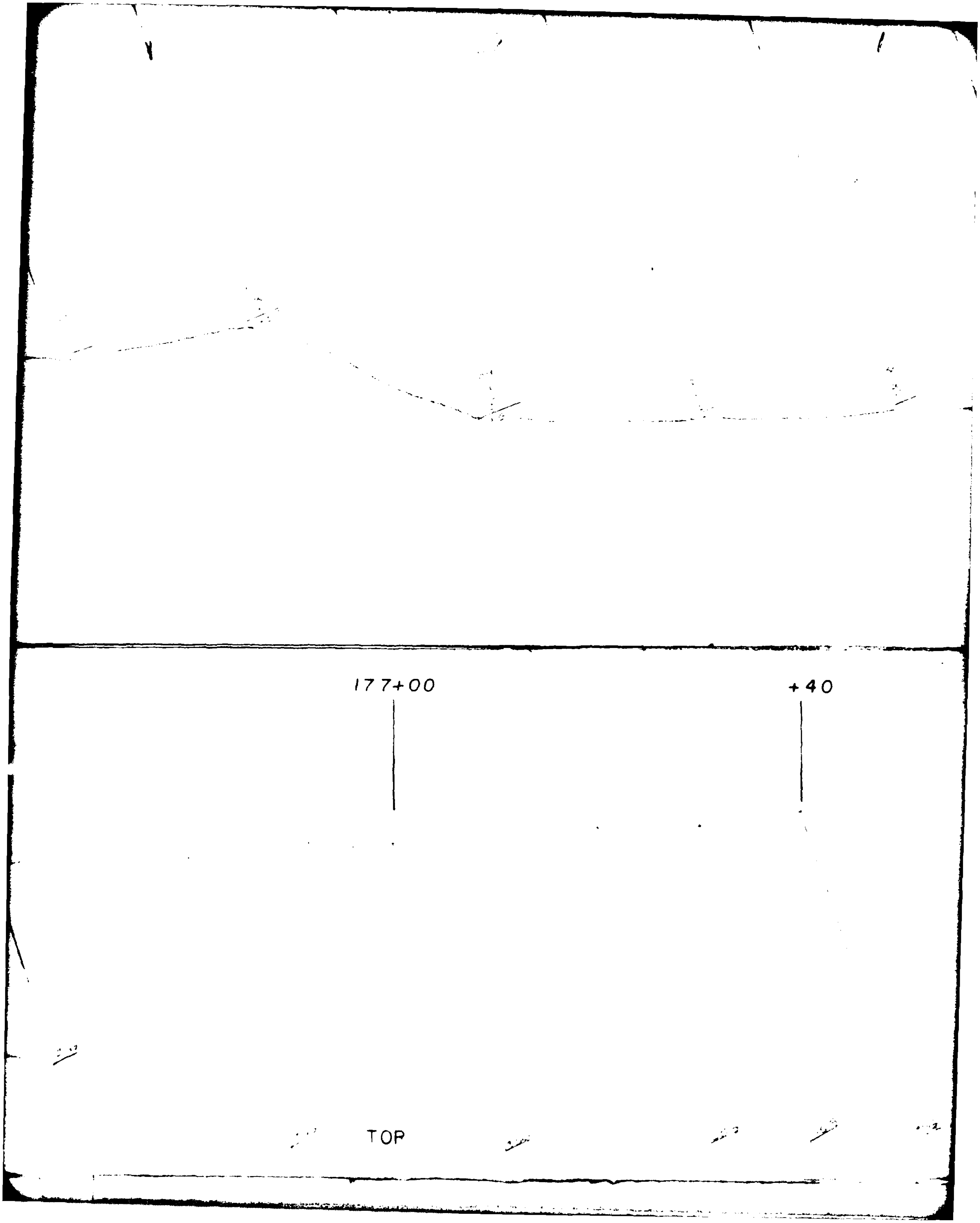


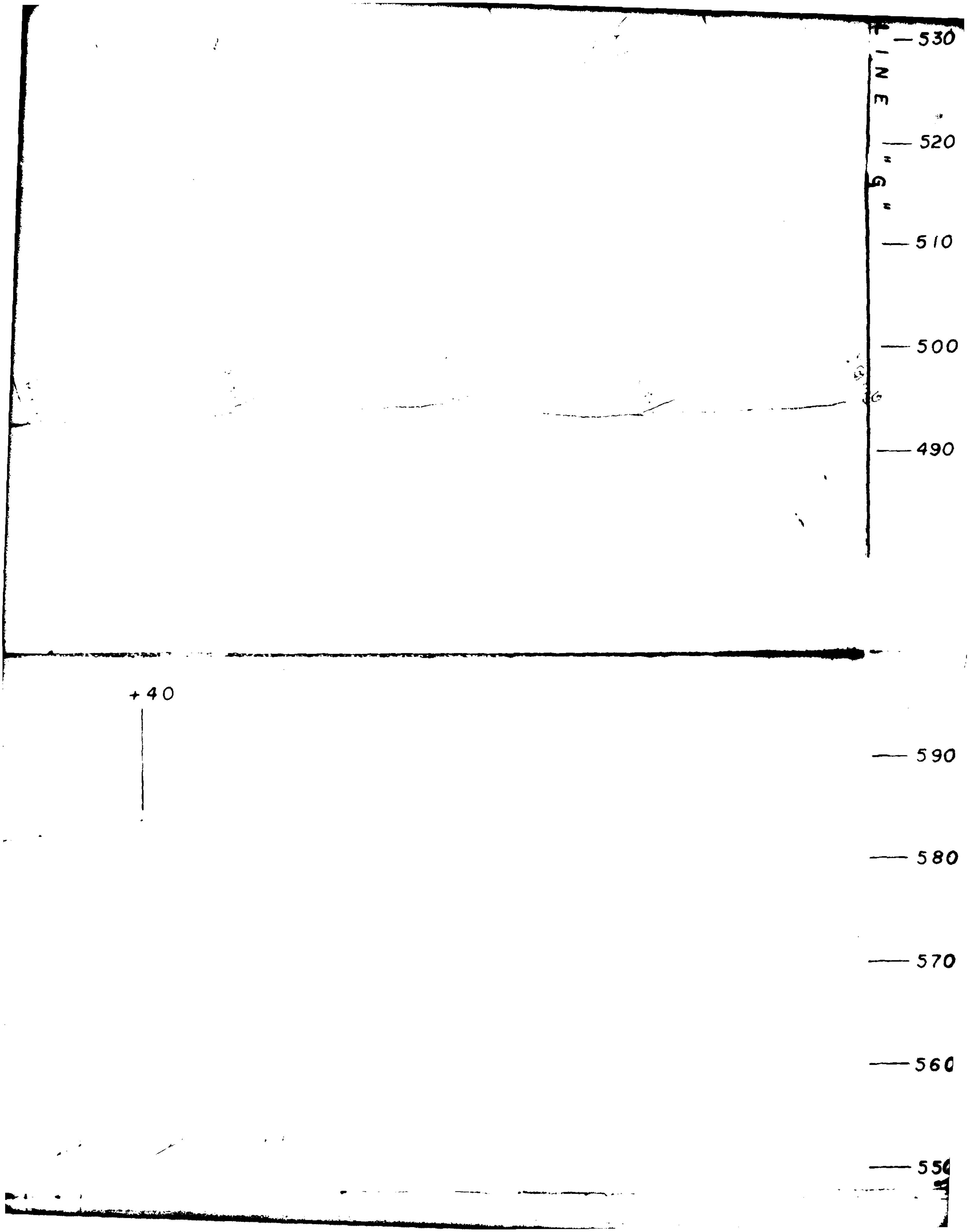
1
5
174+00
1
10
MATCH LINE

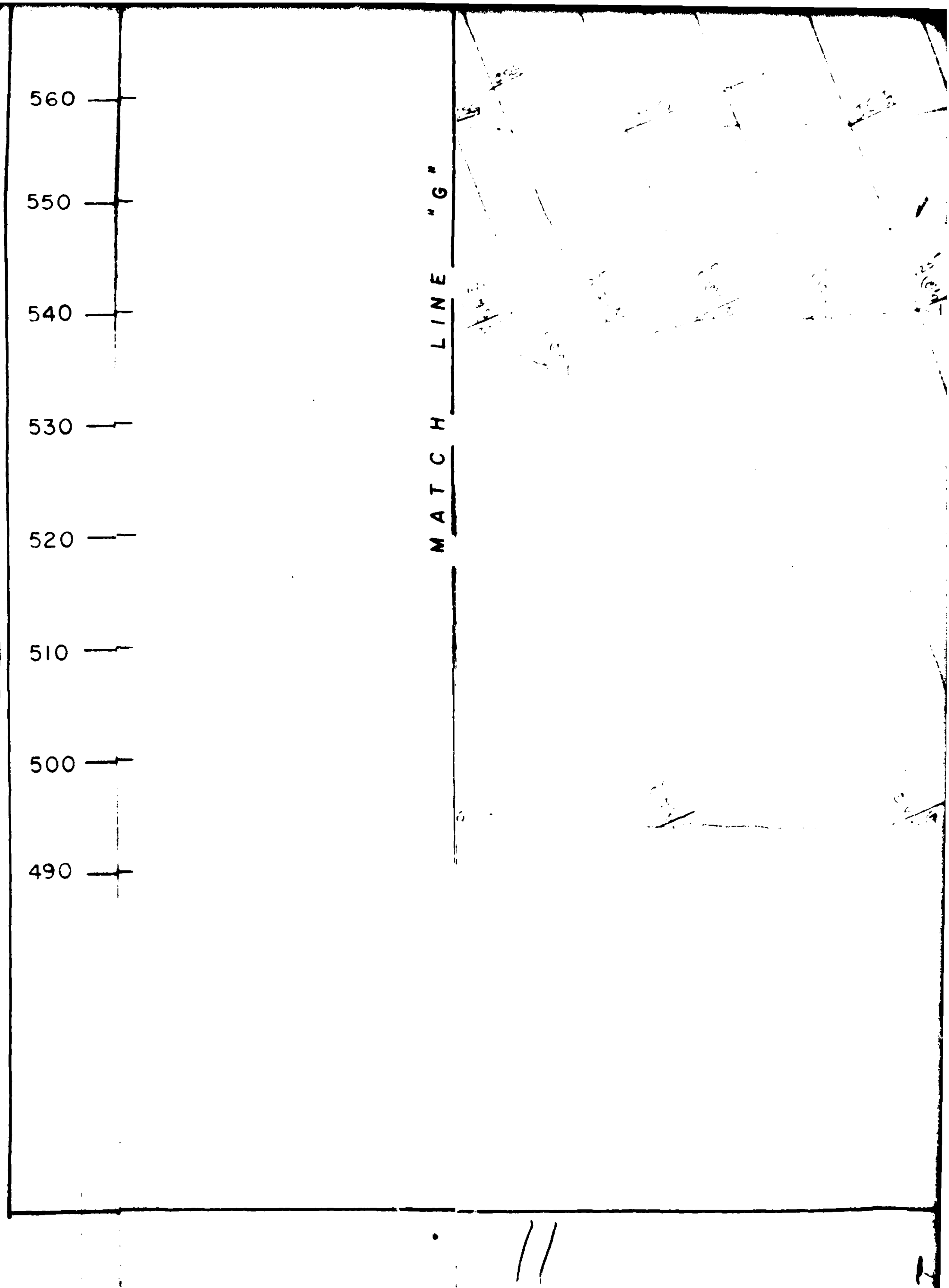

6
MATCH LINE "F"
530
520
510
500
490
175+00
580
570
560
NG
NG
NG
NG
CSG
CSG
CSG
CSG


NG
NG
CSG
CSG
CSG
NP
CSG
TOR
CSG
CSG
CSG

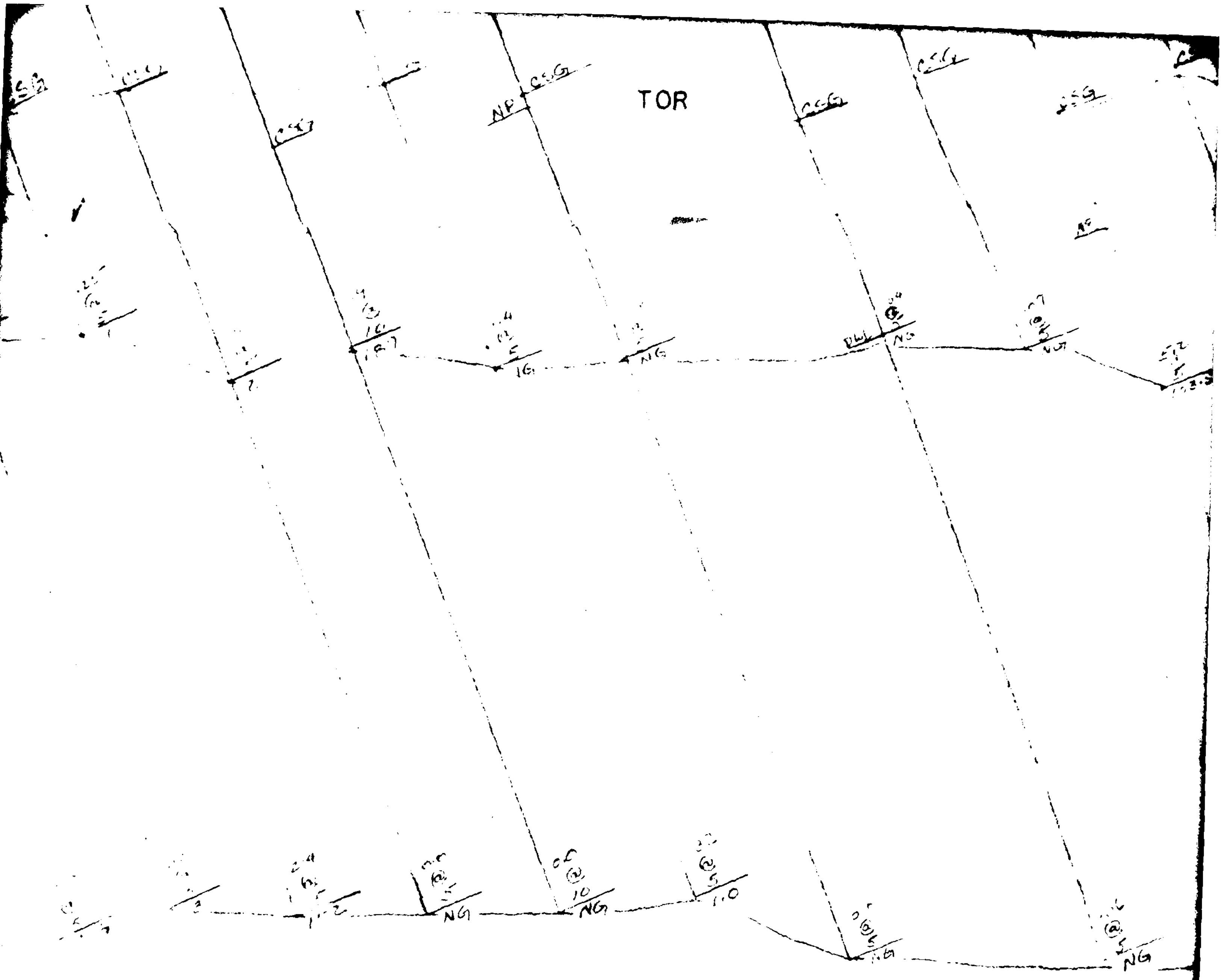

ZONE 2
176+00
ZONE 1

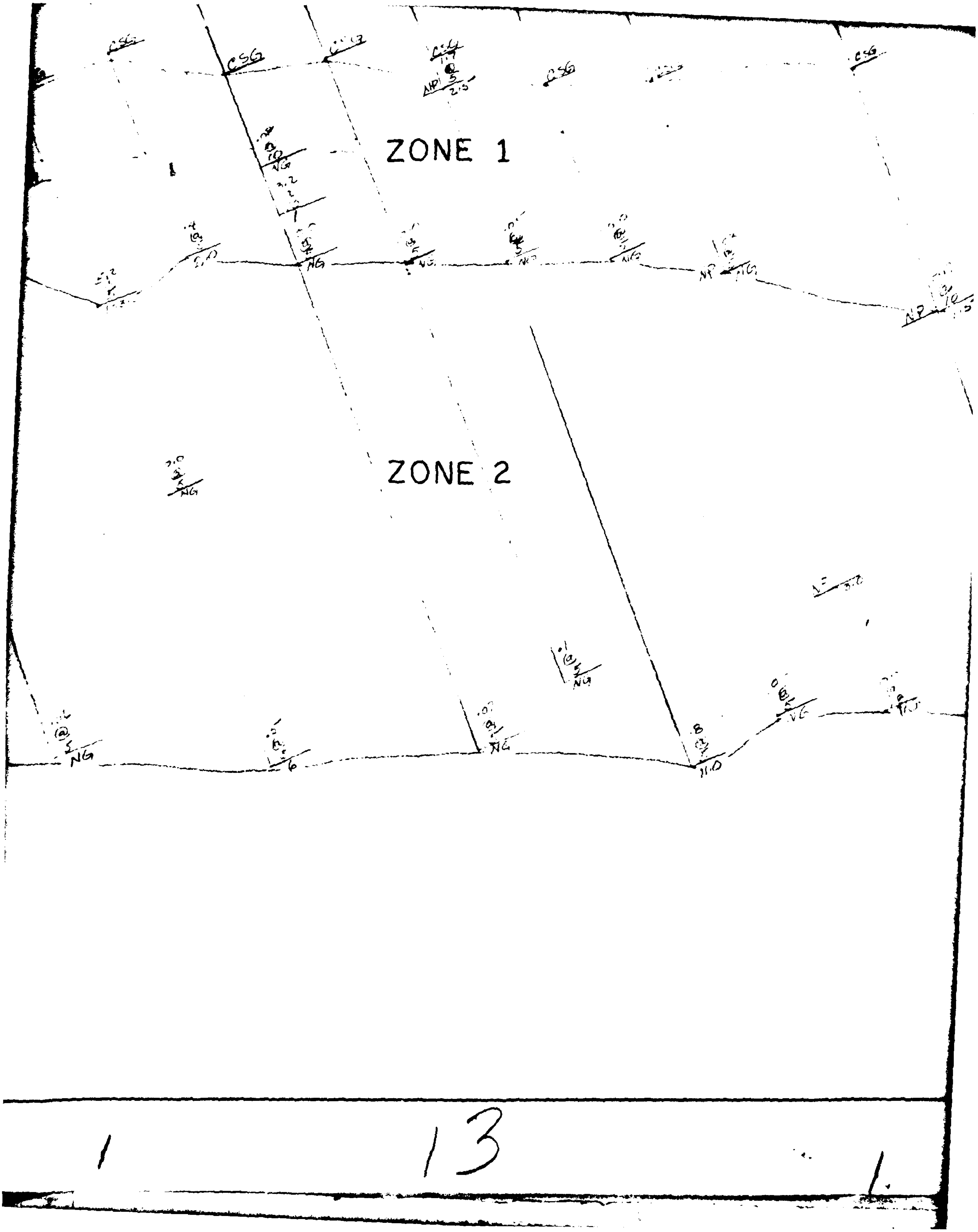

177+00
+40
TOP

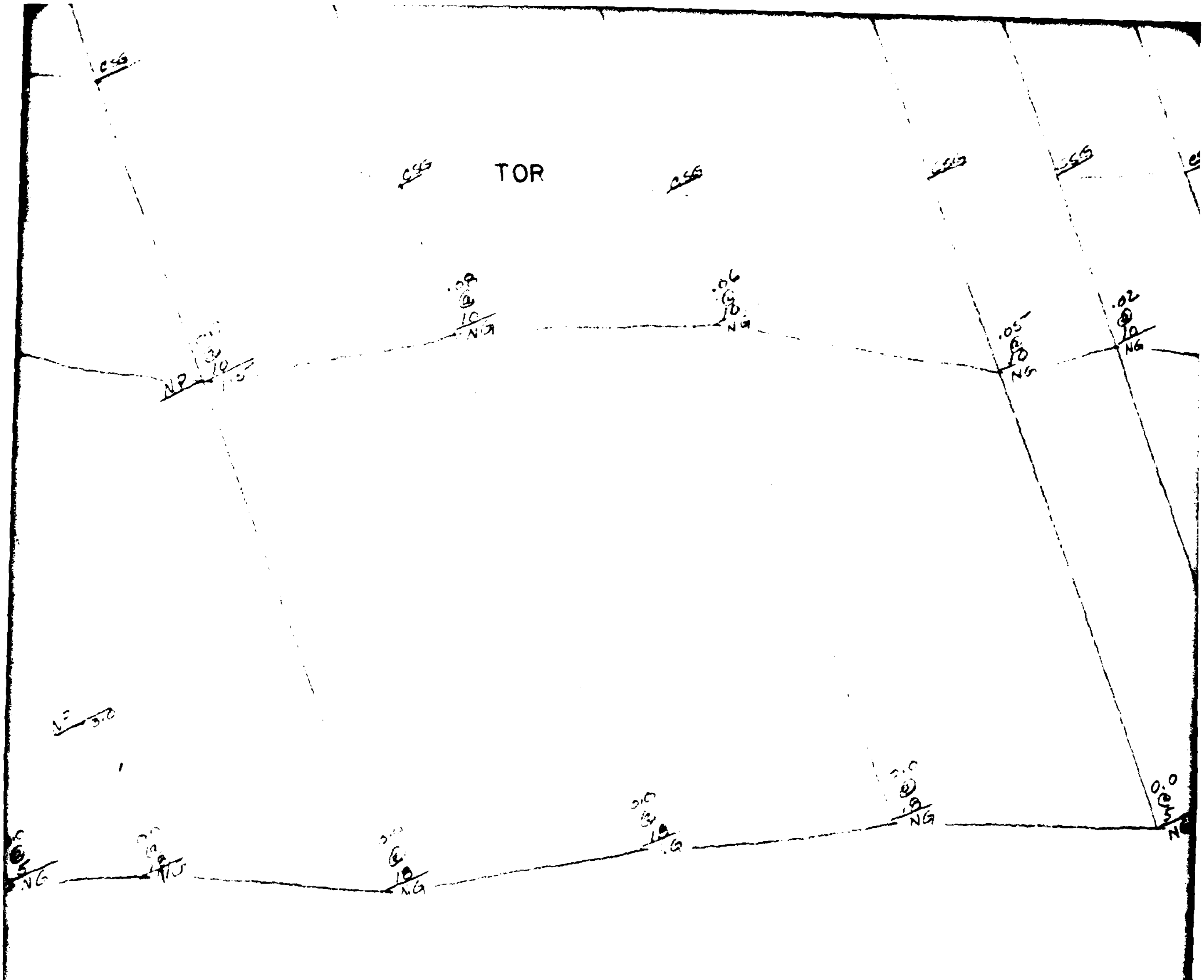

530
LINE "G"
520
510
500
490
+40
590
580
570
560
550



11


TOR
NP
CSG
NG


ZONE 1
ZONE 2
CSG
NG
13



DESIGNED:

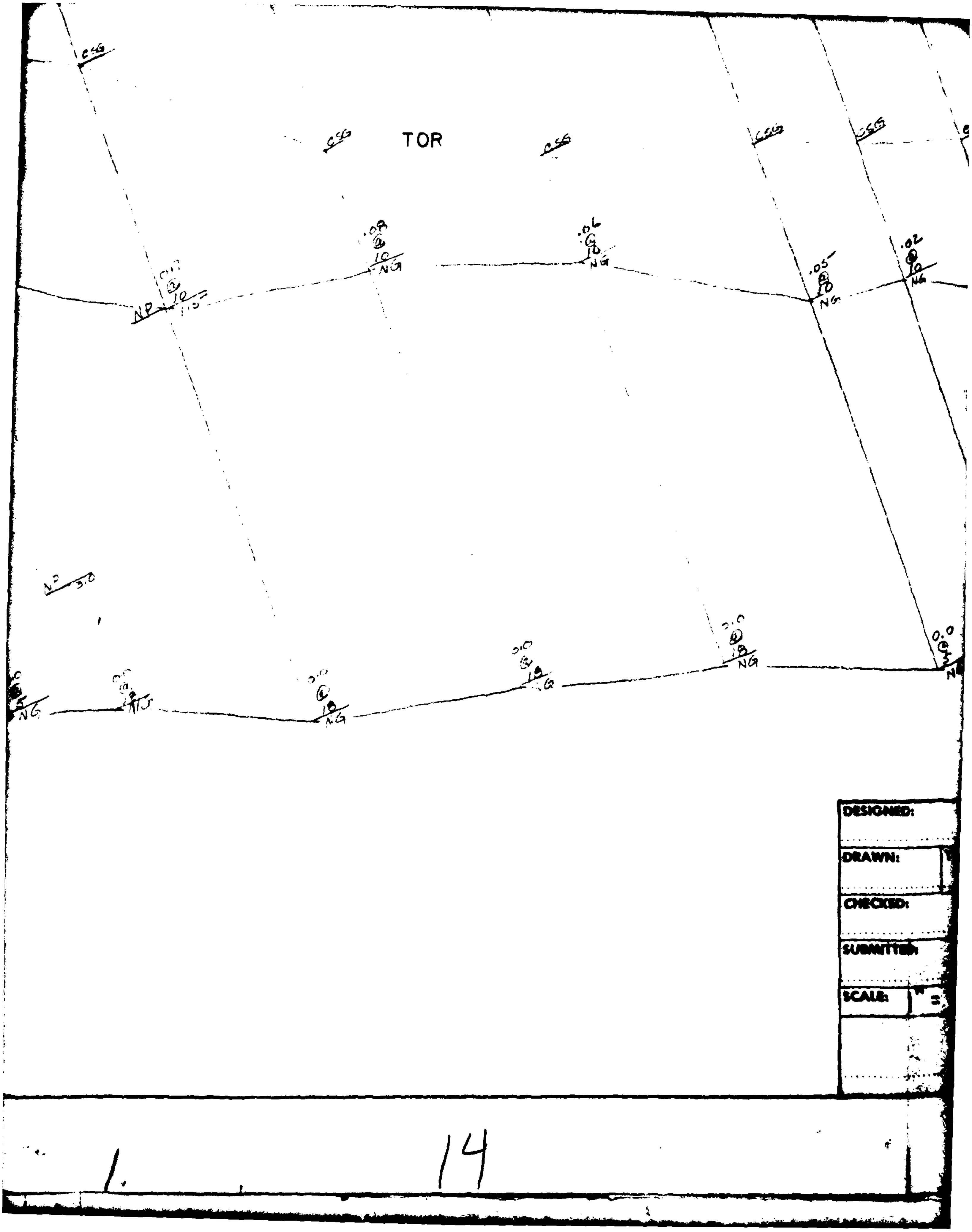
DRAWN:

CHECKED:

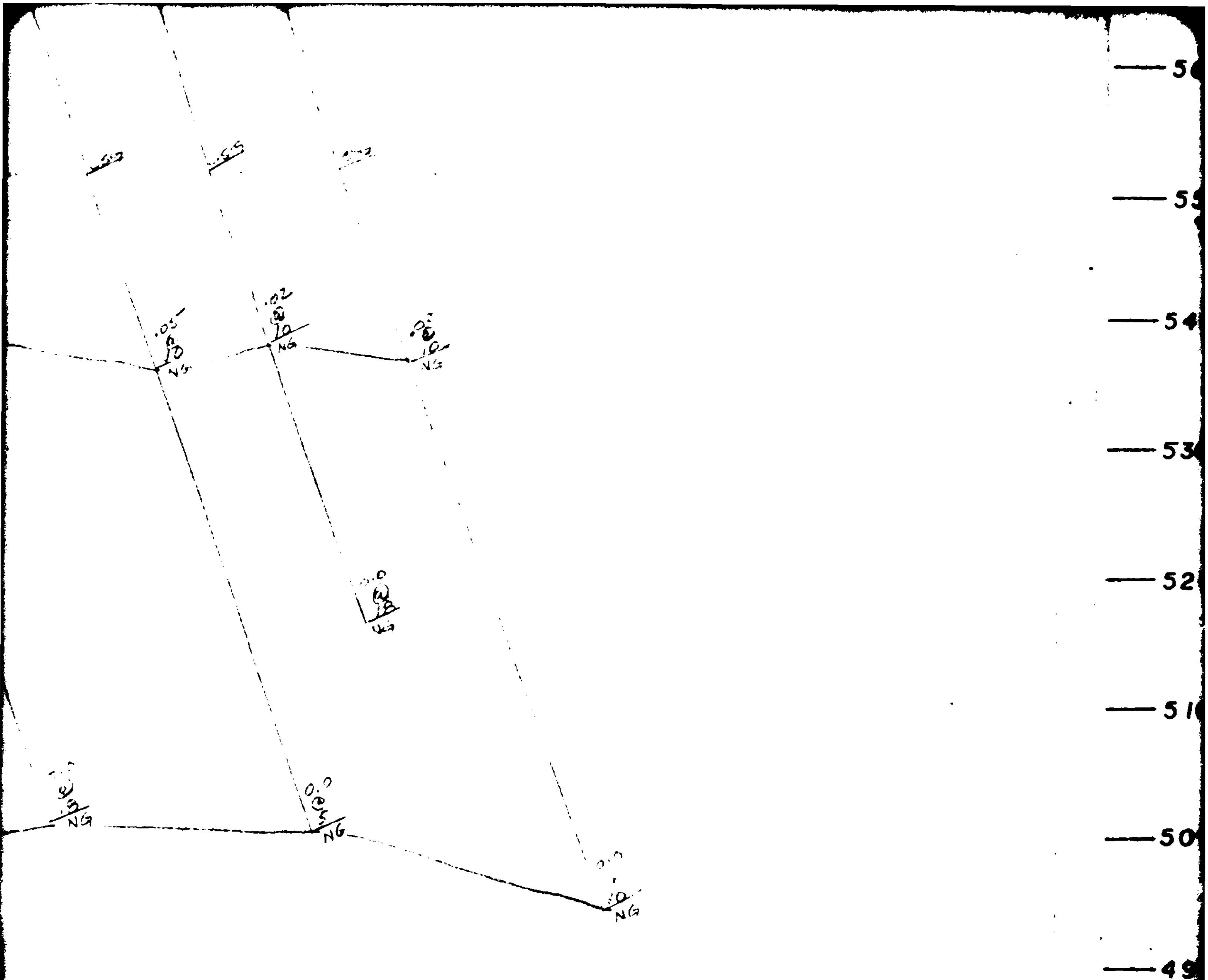
SUBMITTED:

SCALE: 1" =

1. 14



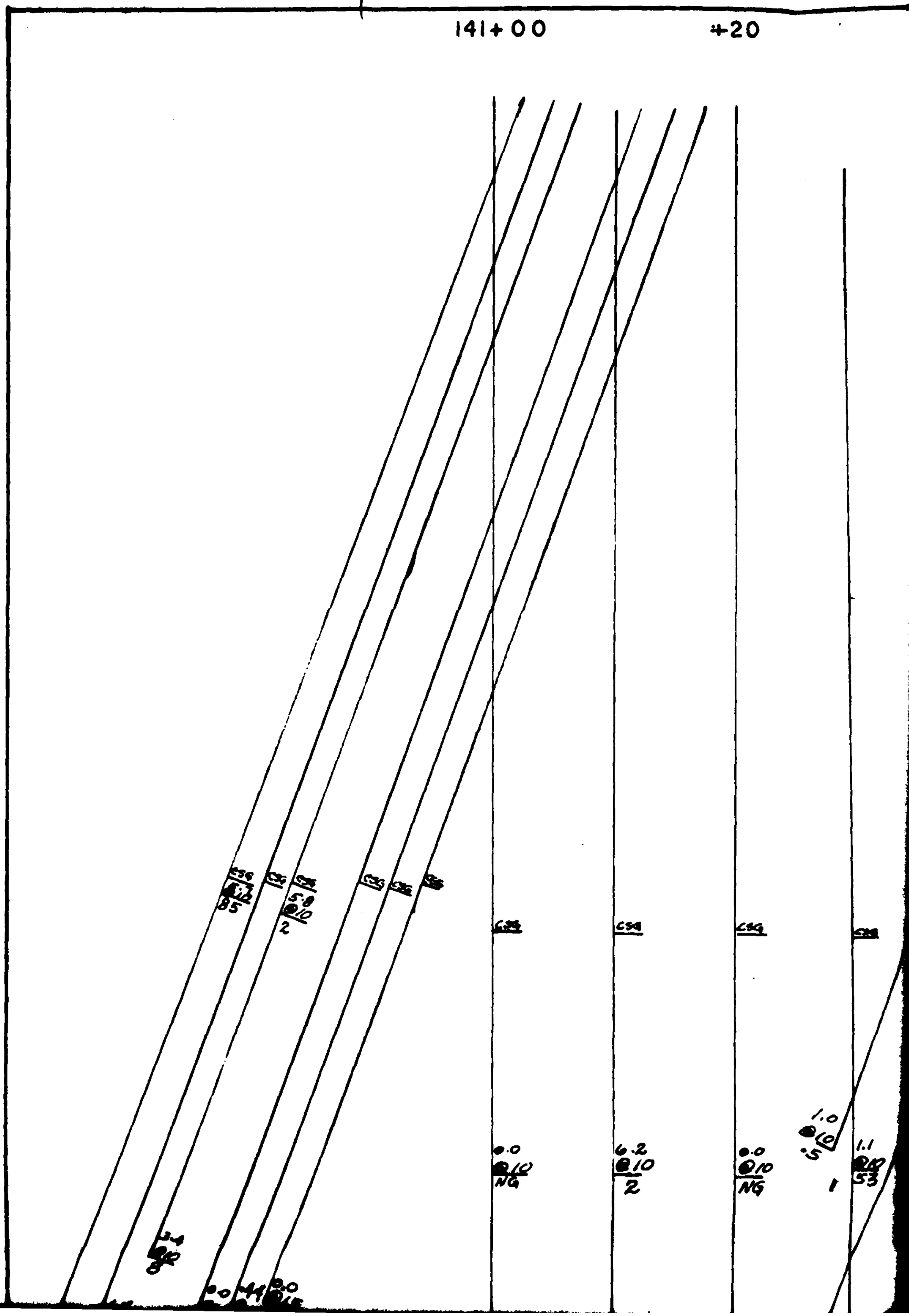
CSG
TOR
CSG
CSG
CSG
.08
10
NG
.06
10
NG
.05
10
NG
.02
10
NG
NP
NP
NG
NG
NG
NG
NG
DESIGNED:
DRAWN:
CHECKED:
SUBMITTED:
SCALE:
1.
14

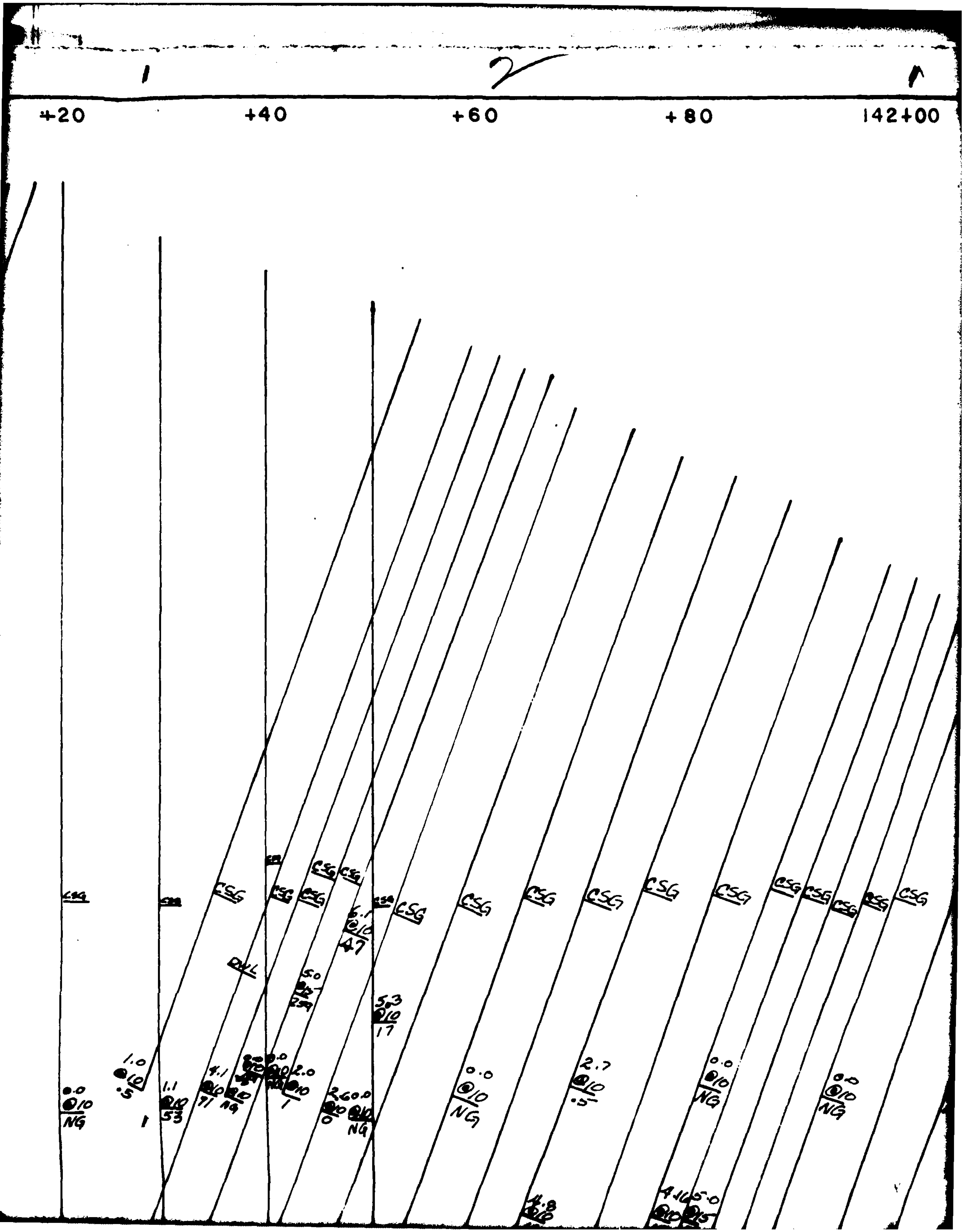


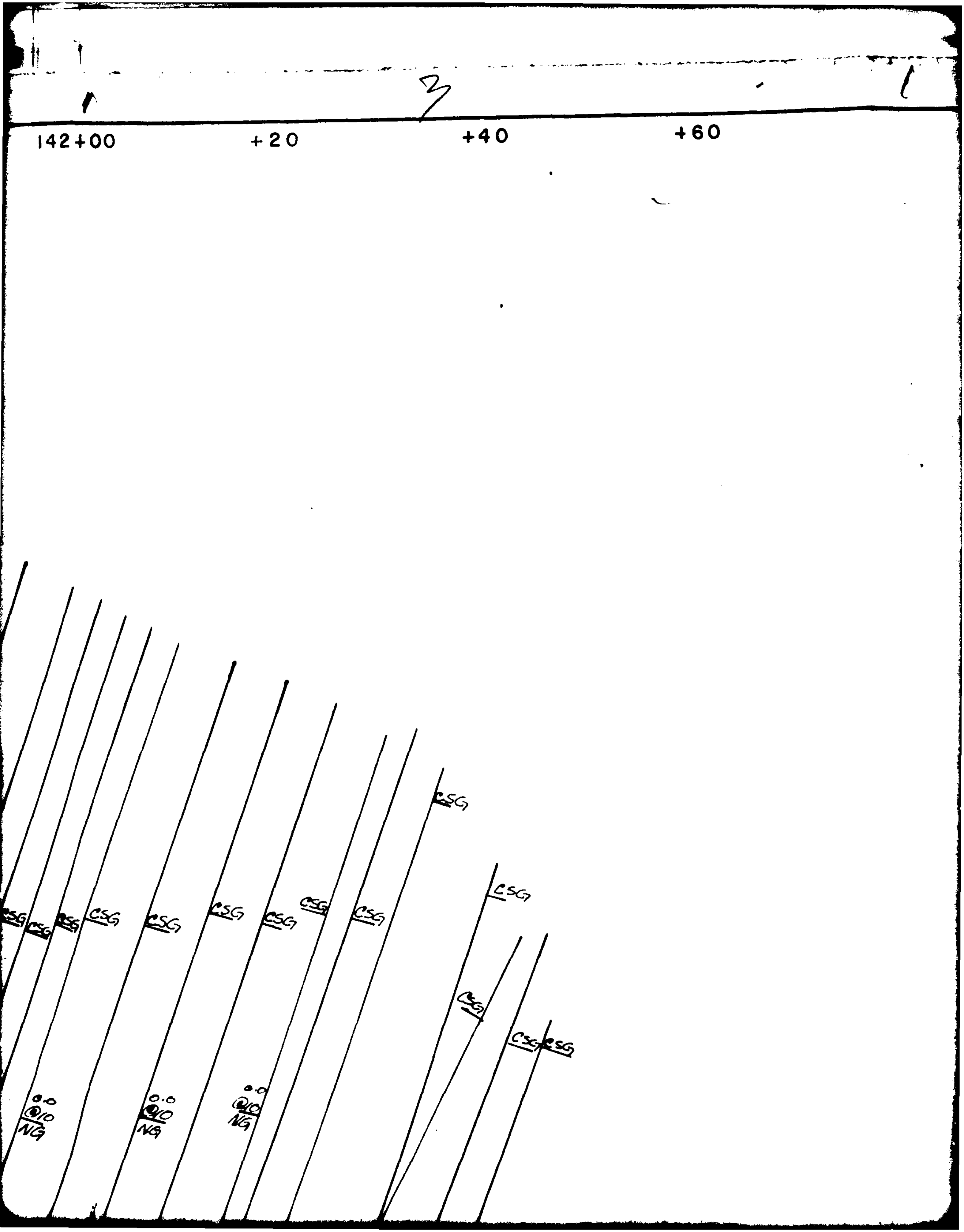
DESIGNED:	PATOKA LAKE
DRAWN: TRACED:	DAM & SPILLWA
CHECKED:	GROUT PROFILE DOWN STREAM LI
SUBMITTED:	STATION 171+00 to 174+70 STATION 17
SCALE: 1" = 10' NATURAL	DATE:
	DRAWING NU FIGURE G-

15

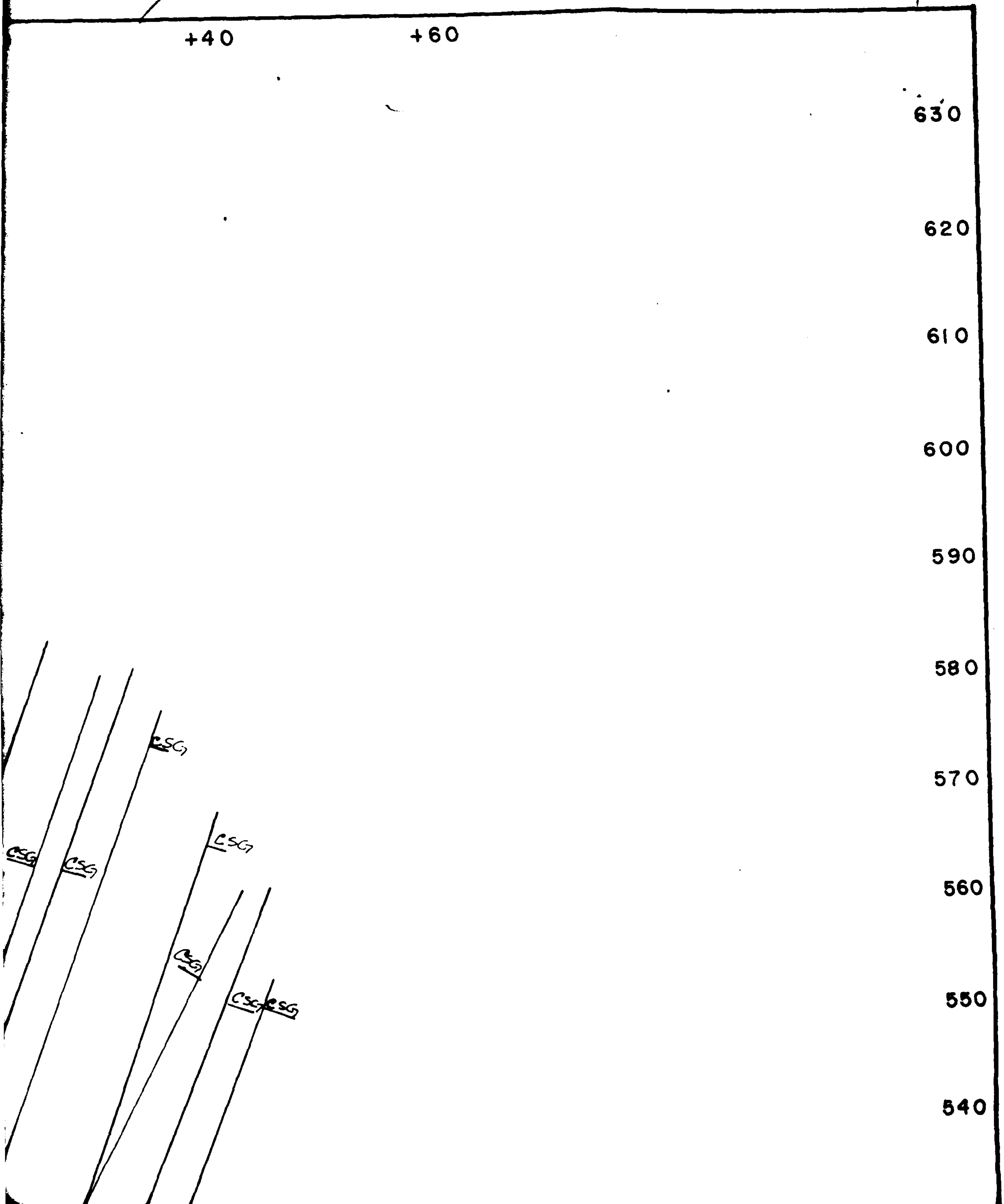
1

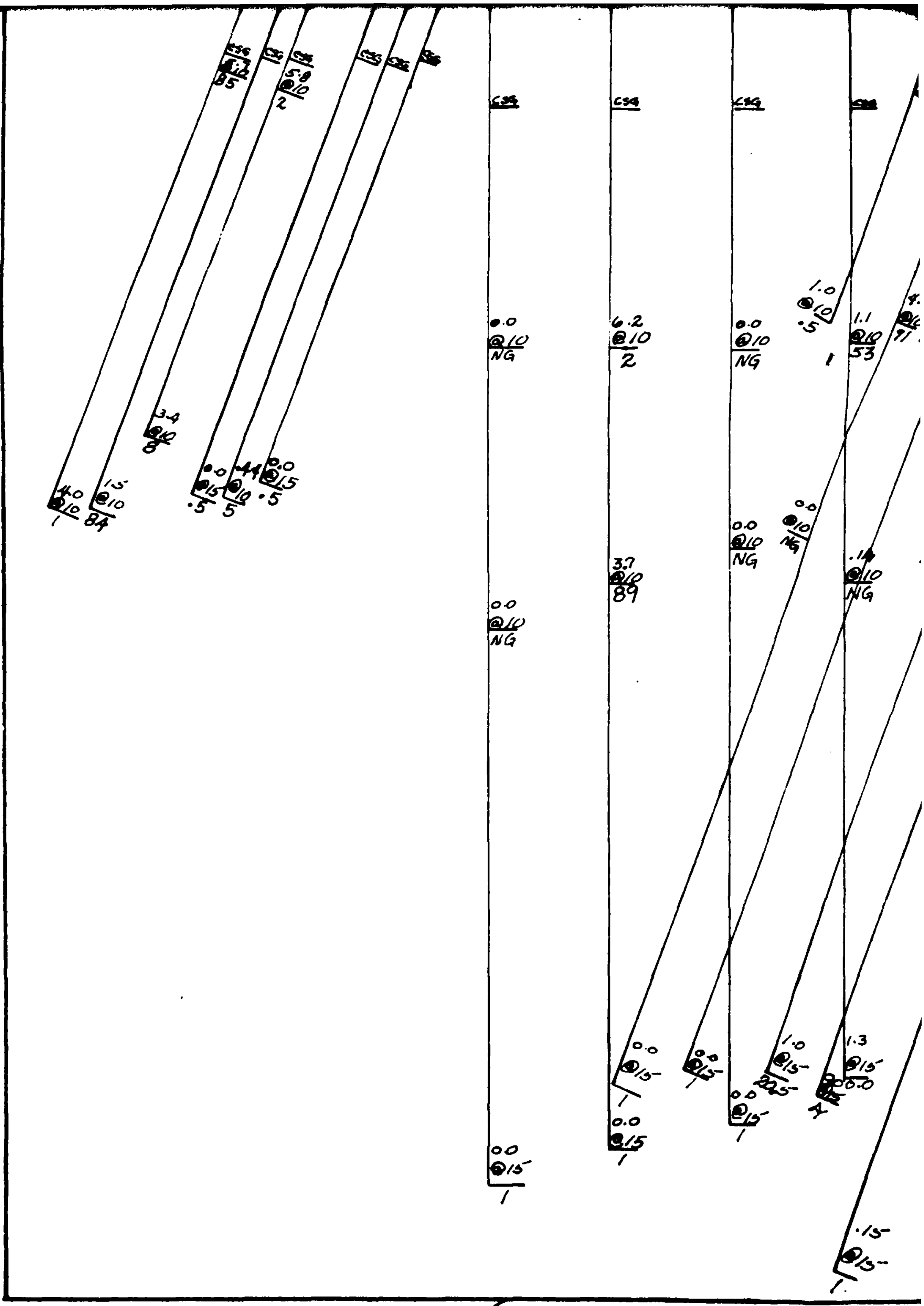



+20
+40
+60
+80
142+00
CSG
CSG
CSG
CSG
CSG
CSG
CSG
CSG
CSG
CSG
CSG
CSG
CSG
CSG
CSG
CSG
CSG
DHL
0.0 @10 NG
1.0 @10 .5
1.1 @10 53
4.1 @10 91
2.0 @10 1
5.3 @10 17
0.0 @10 NG
2.7 @10 .5
0.0 @10 NG
0.0 @10 NG

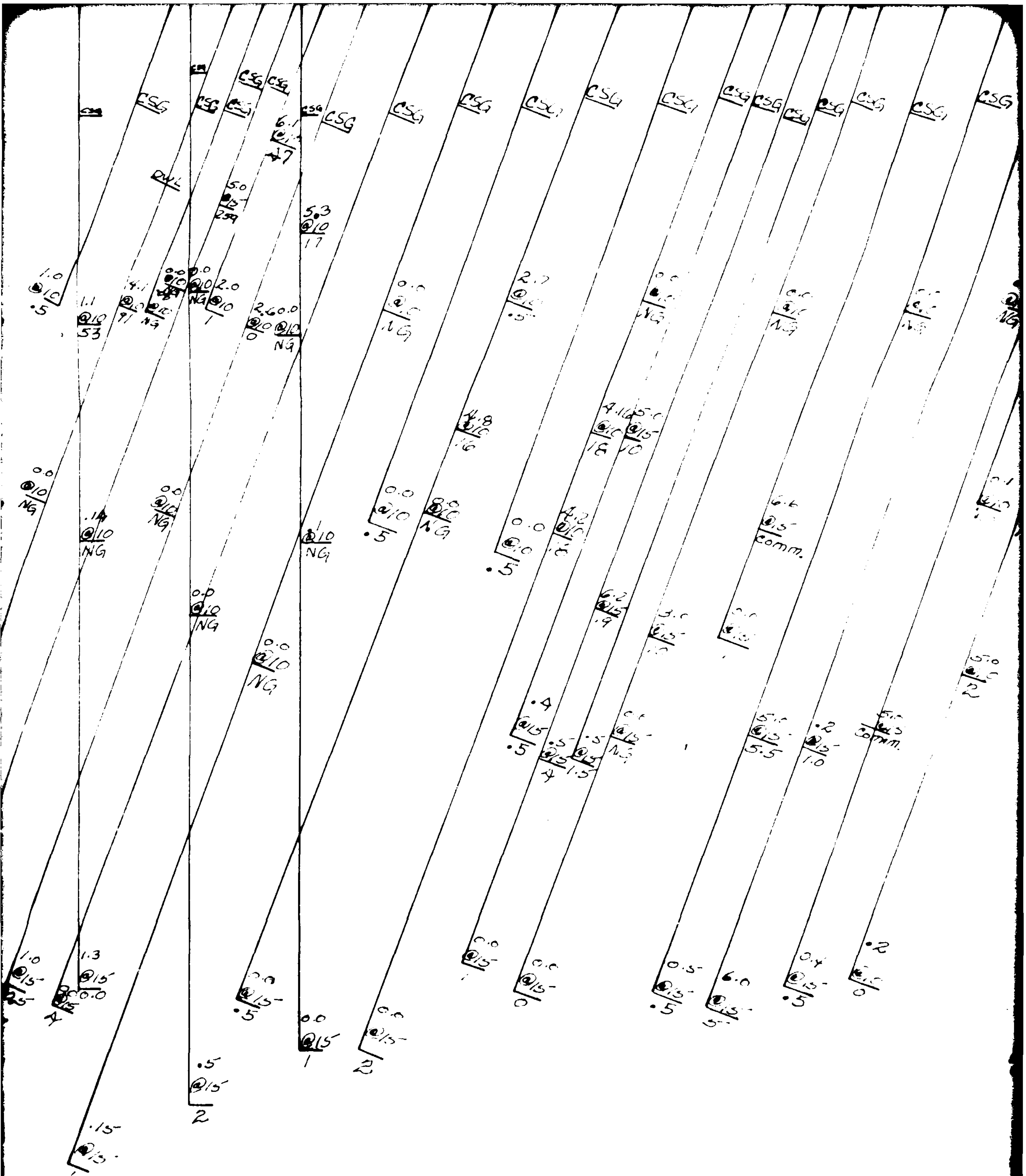

142+00
+20
+40
+60
CSG
0.0
@10
NG

3 1 4

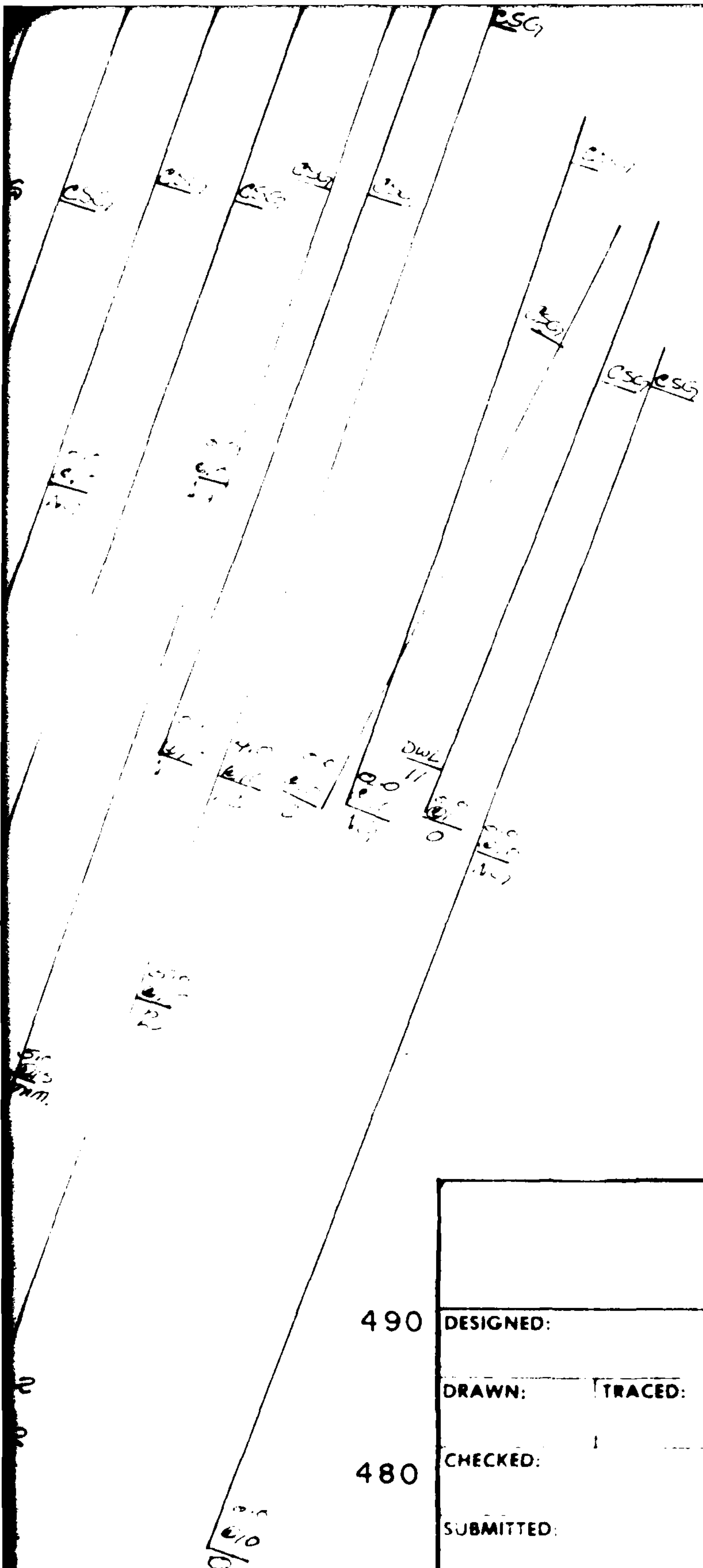

+40
+60
630
620
610
600
590
580
570
560
550
540
CSG
CSG
CSG
CSG
CSG
CSG
CSG
CSG



CSG
0.0 @10 NG
6.2 @10 2
0.0 @10 NG
1.0 @10 .5
1.1 @10 53
3.4 @10 8
4.0 @10 1
.15 @10 84
0.0 @15 .5
.44 @10 5
0.0 @15 .5
0.0 @10 NG
3.7 @10 89
0.0 @10 NG
0.0 @15 1
0.0 @15 1
0.0 @15 1
0.0 @15 1
1.0 @15
1.3 @15
.15 @15 1

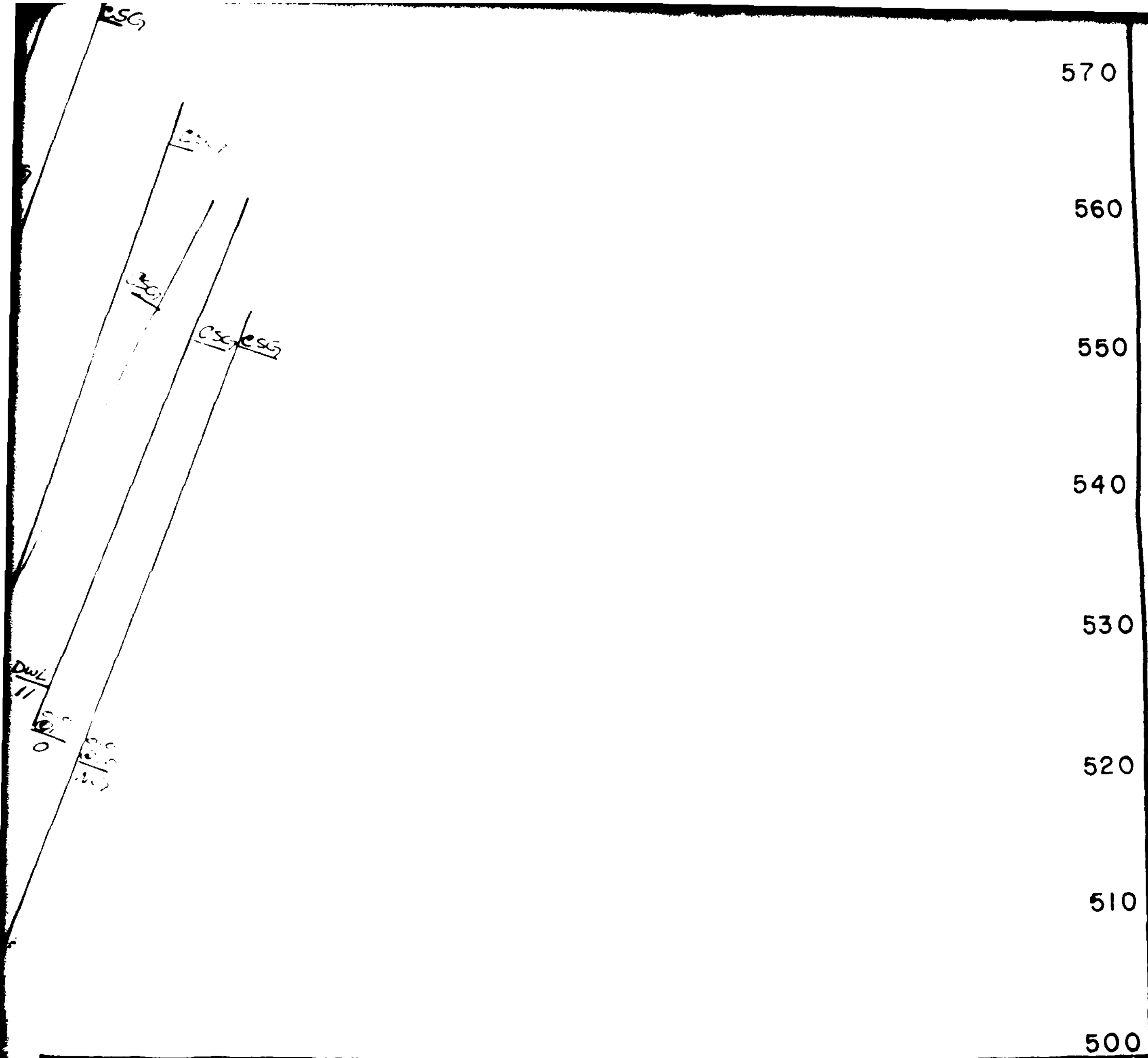


1 6



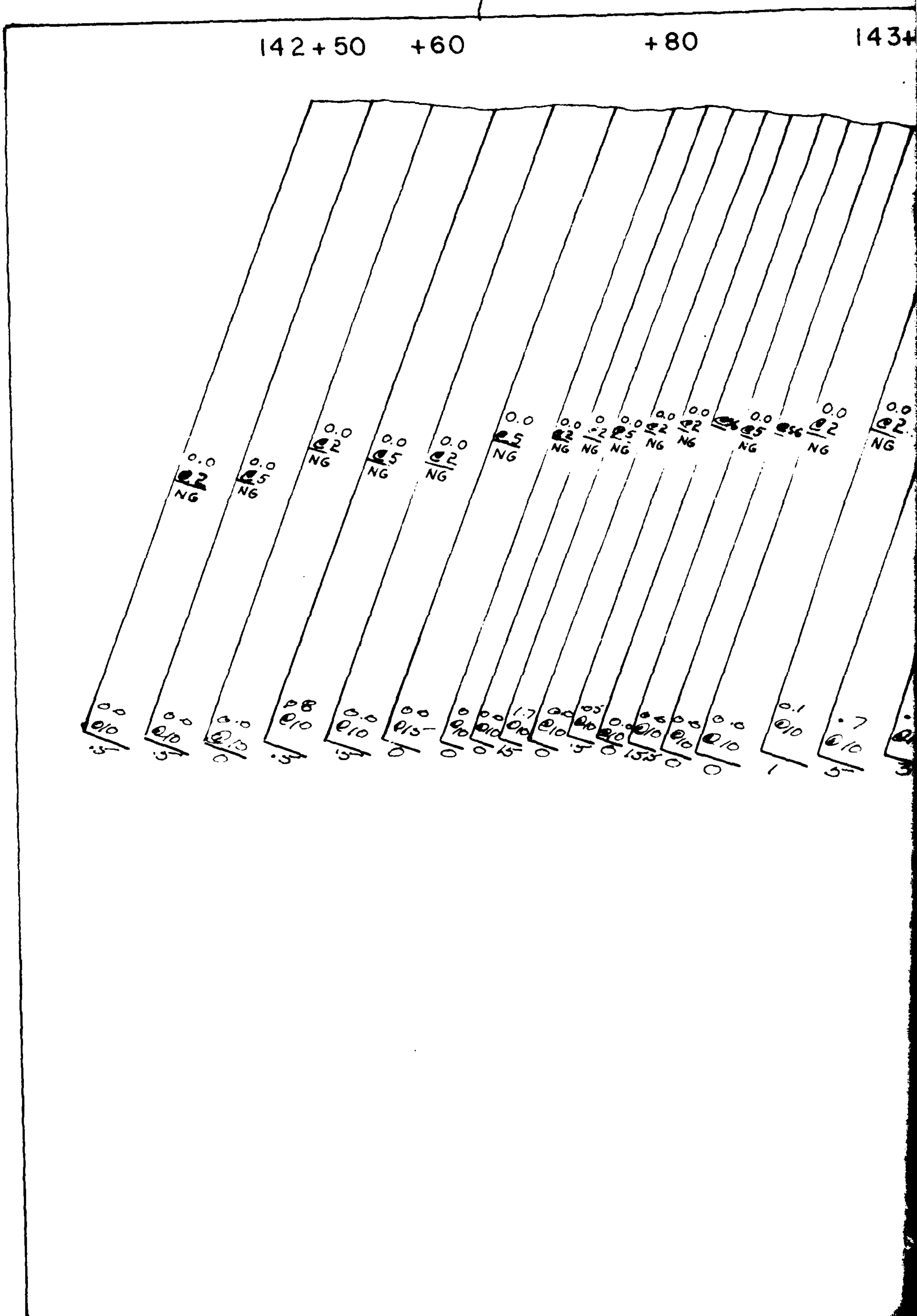
U. S. ARMY ENGINEER DISTRICT, LOUISVILLE
CORPS OF ENGINEERS
LOUISVILLE, KENTUCKY

DESIGNED:	PATOKA LAK
DRAWN: TRACED:	DAM & SPILLWA
CHECKED:	GROUT PROFILE
SUBMITTED:	CENTERLINE STA. 141+00 to STA. 14
SCALE: 1" = 10' NATURAL	DATE:
	DRAWING NUMB FIGURE C

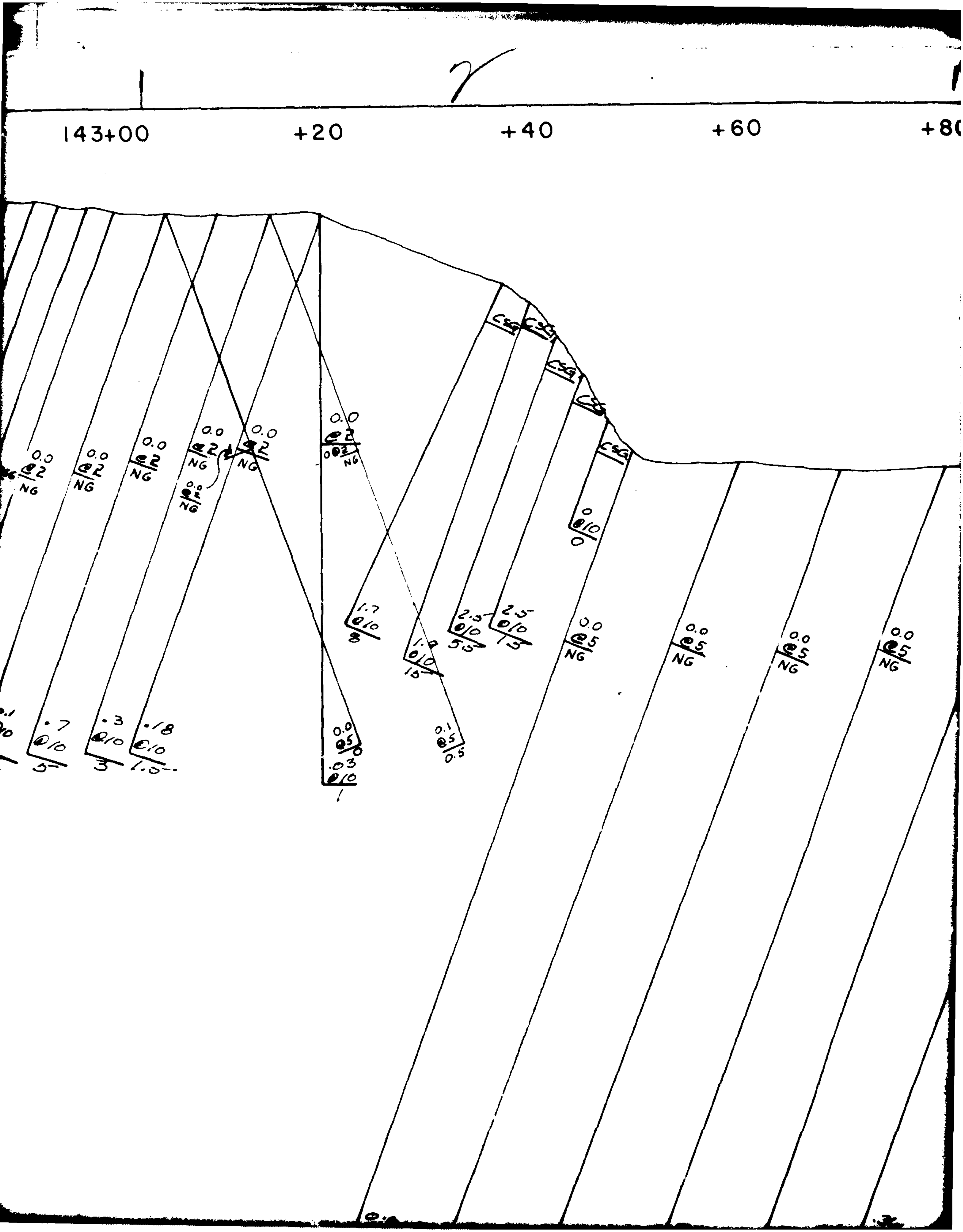


U. S. ARMY ENGINEER DISTRICT, LOUISVILLE
CORPS OF ENGINEERS
LOUISVILLE, KENTUCKY

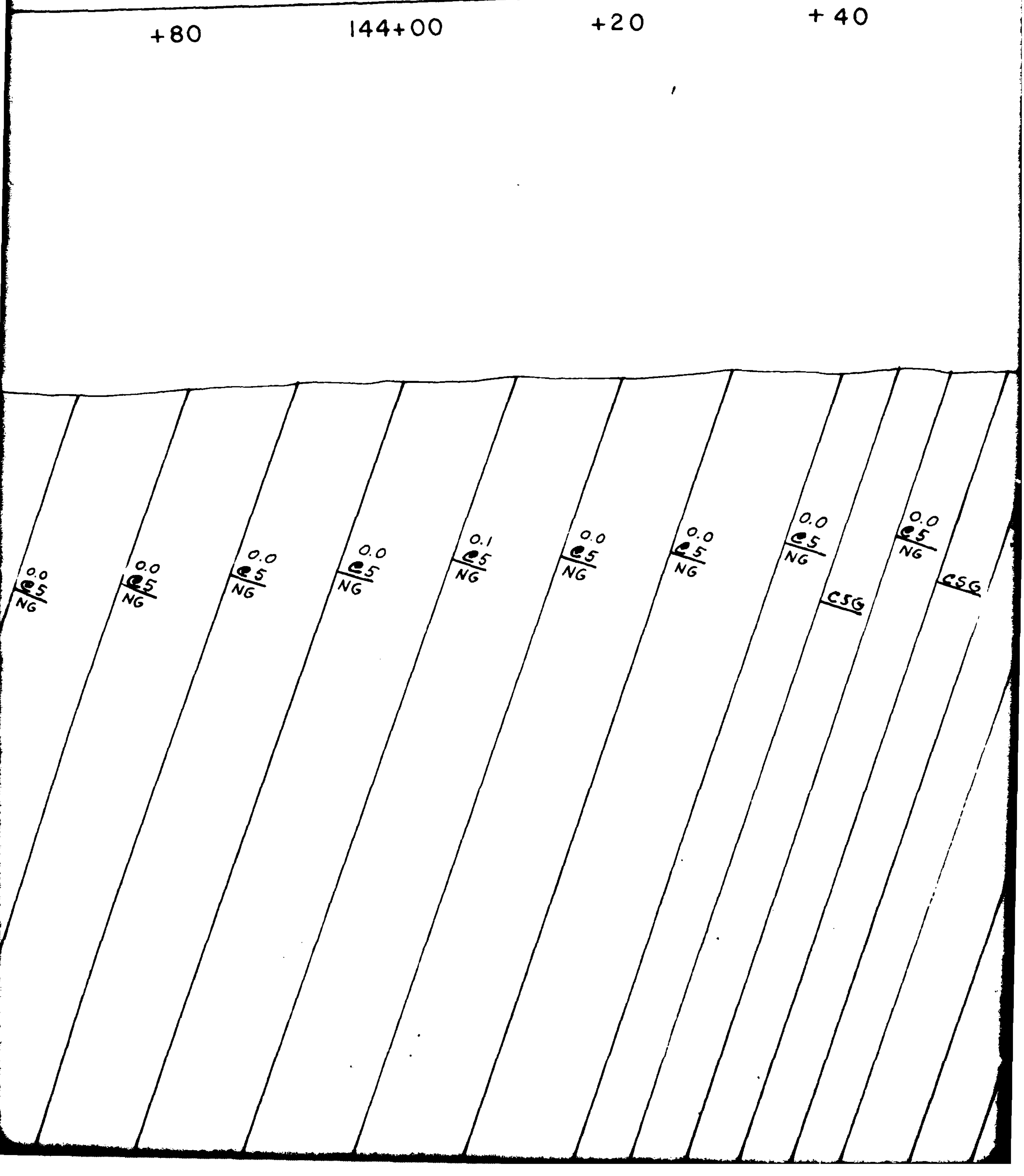
DESIGNED:	PATOKA LAKE DAM & SPILLWAY GROUT PROFILE
DRAWN: TRACED:	
CHECKED:	
SUBMITTED:	CENTERLINE STA. 141+00 to STA. 142+50
SCALE: 1" = 10' NATURAL	DATE:
	DRAWING NUMBER FIGURE C-6



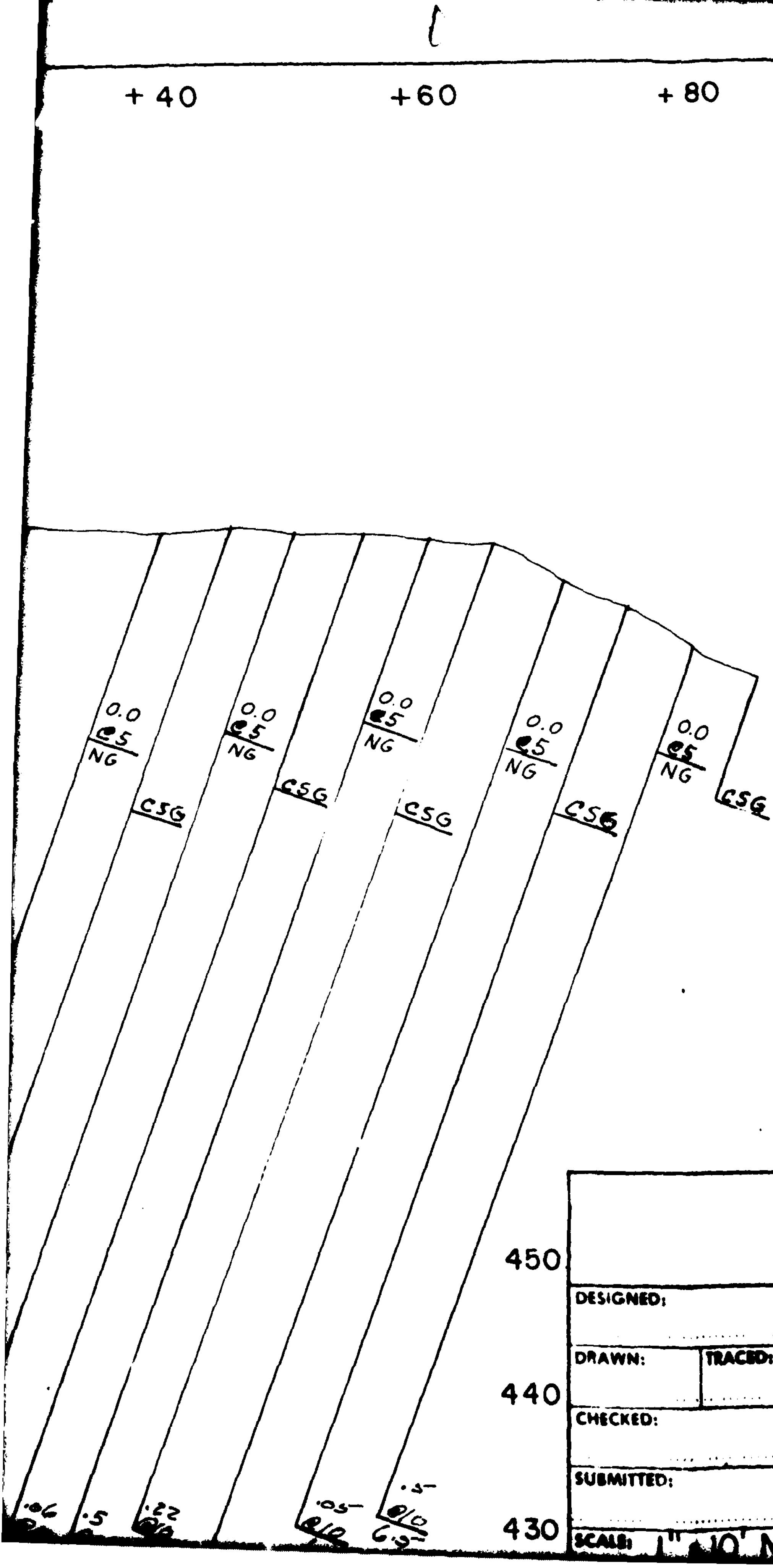
142+50
+60
+80
143+


143+00
+20
+40
+60
+80
CSG
0.0
@2
NG
0.0
@5
NG
1.7
@10
8
2.5
@10
5.5
2.5
@10
1.5
0
@10
0
0.0
@5
.03
@10
1
0.1
@5
0.5
.7
@10
5
.3
@10
3
.18
@10
1.5

1 3



+80
144+00
+20
+40
0.0 @5 NG
0.0 @5 NG
0.0 @5 NG
0.0 @5 NG
0.1 @5 NG
0.0 @5 NG
0.0 @5 NG
0.0 @5 NG
CSG
0.0 @5 NG
CSG

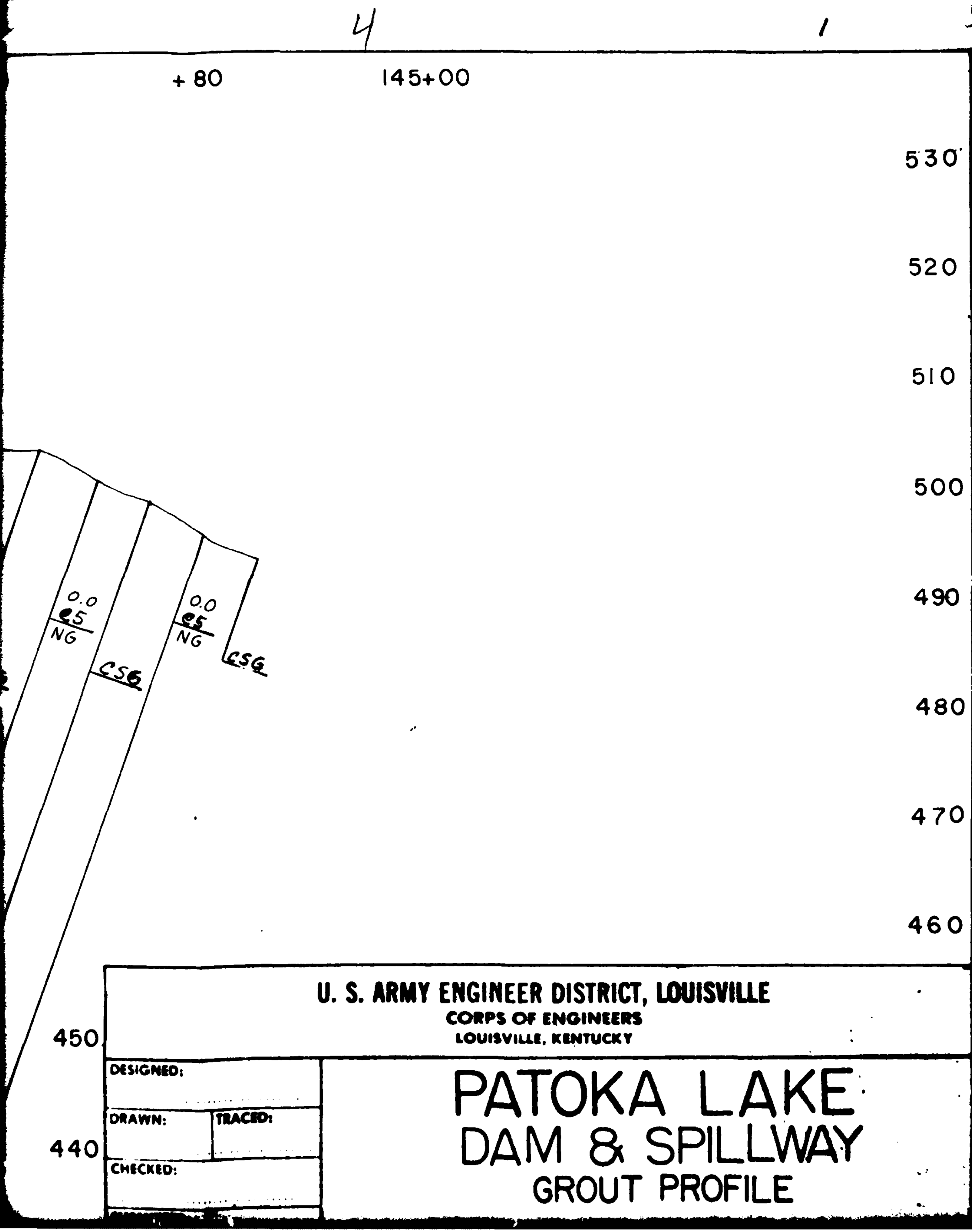


U. S. ARMY ENGINEER DISTRICT,
CORPS OF ENGINEERS
LOUISVILLE, KENTUCKY

DESIGNED:
DRAWN:
TRACED:
CHECKED:
SUBMITTED:

PATOKA
DAM &
GROUT
CENTERLINE STA.

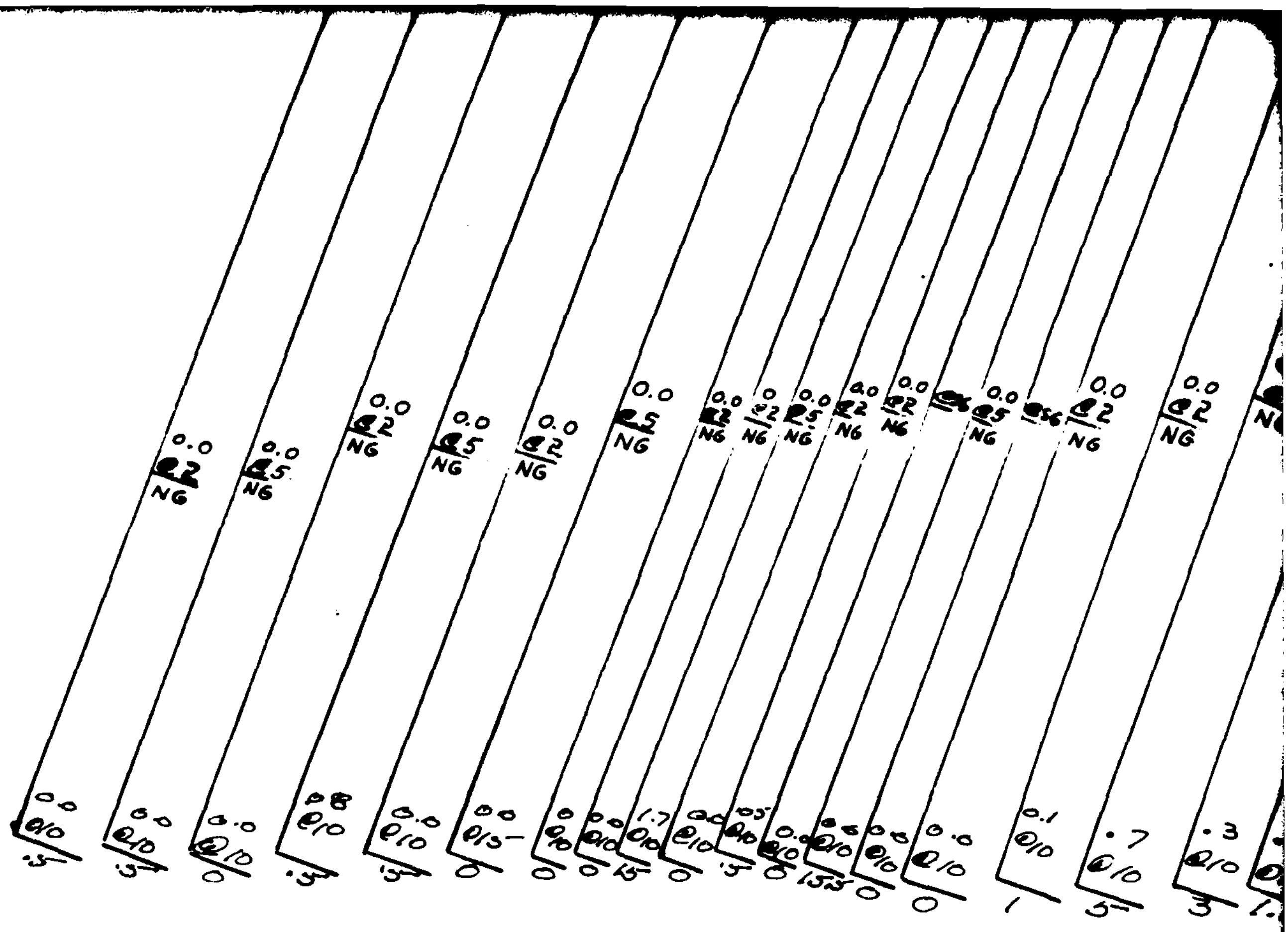
SCALE: 1" = 10' NATURAL
DATE:



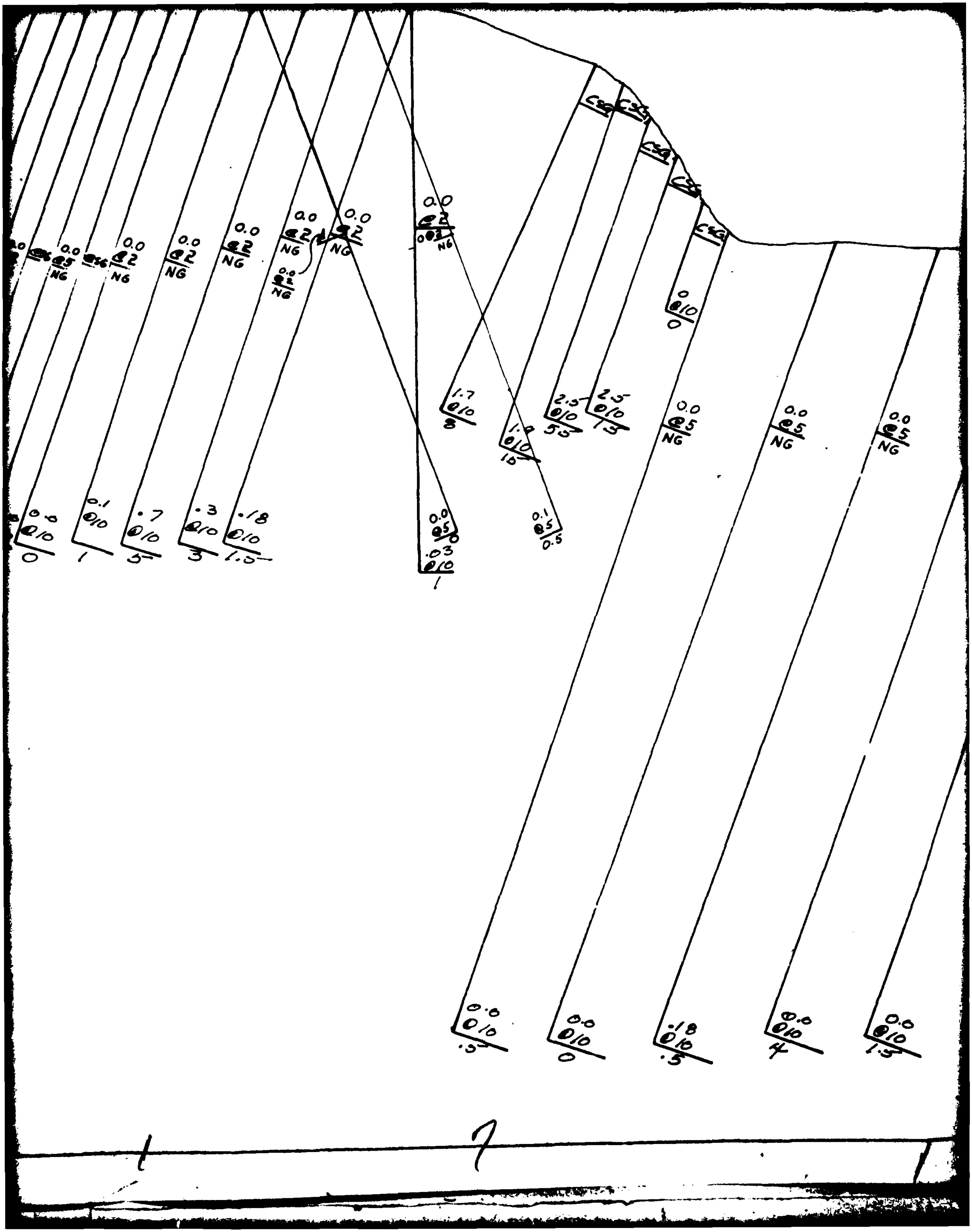
U. S. ARMY ENGINEER DISTRICT, LOUISVILLE
CORPS OF ENGINEERS
LOUISVILLE, KENTUCKY

DESIGNED:
DRAWN: TRACED:
CHECKED:

PATOKA LAKE
DAM & SPILLWAY
GROUT PROFILE



6



CSG
CSG
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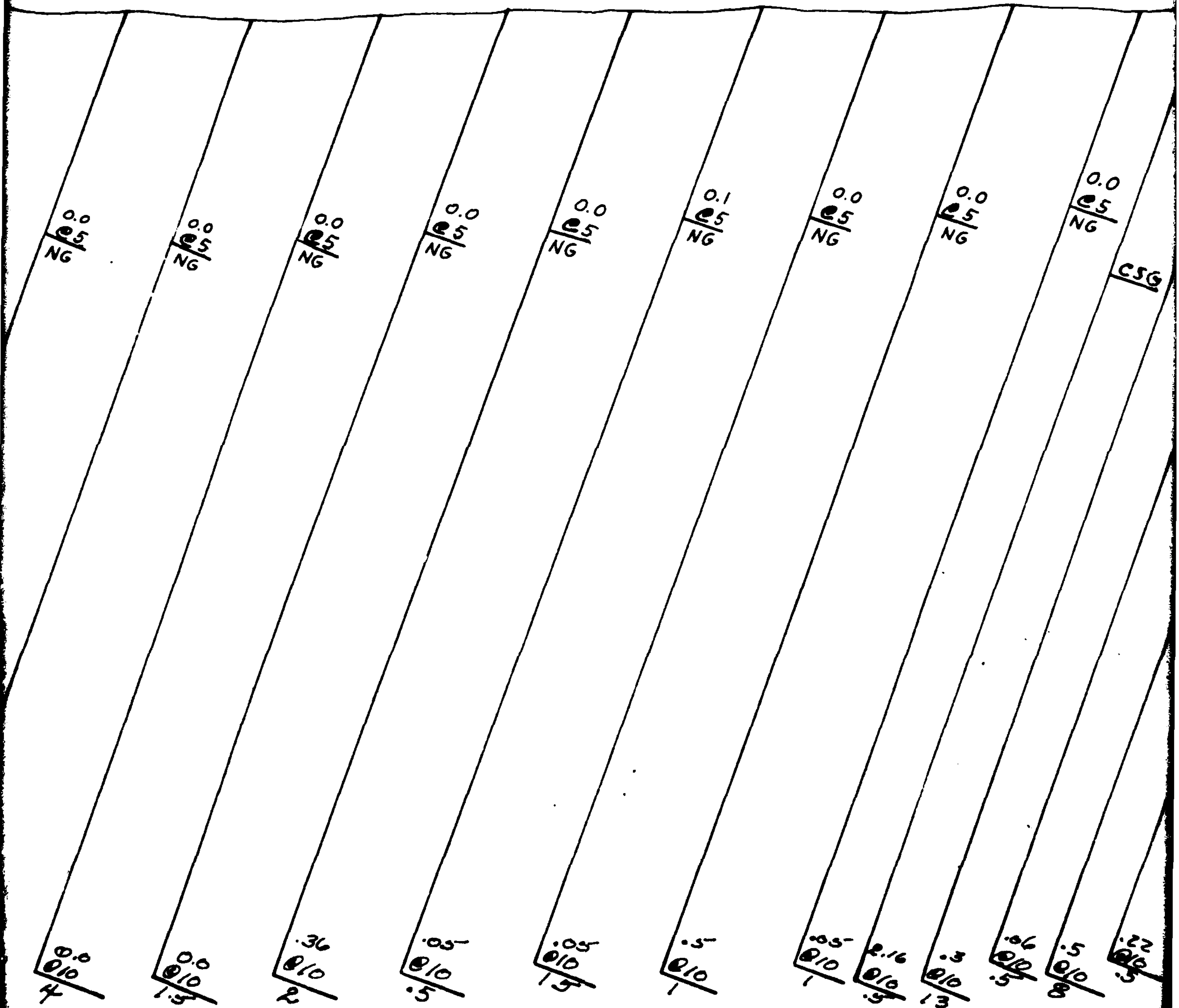
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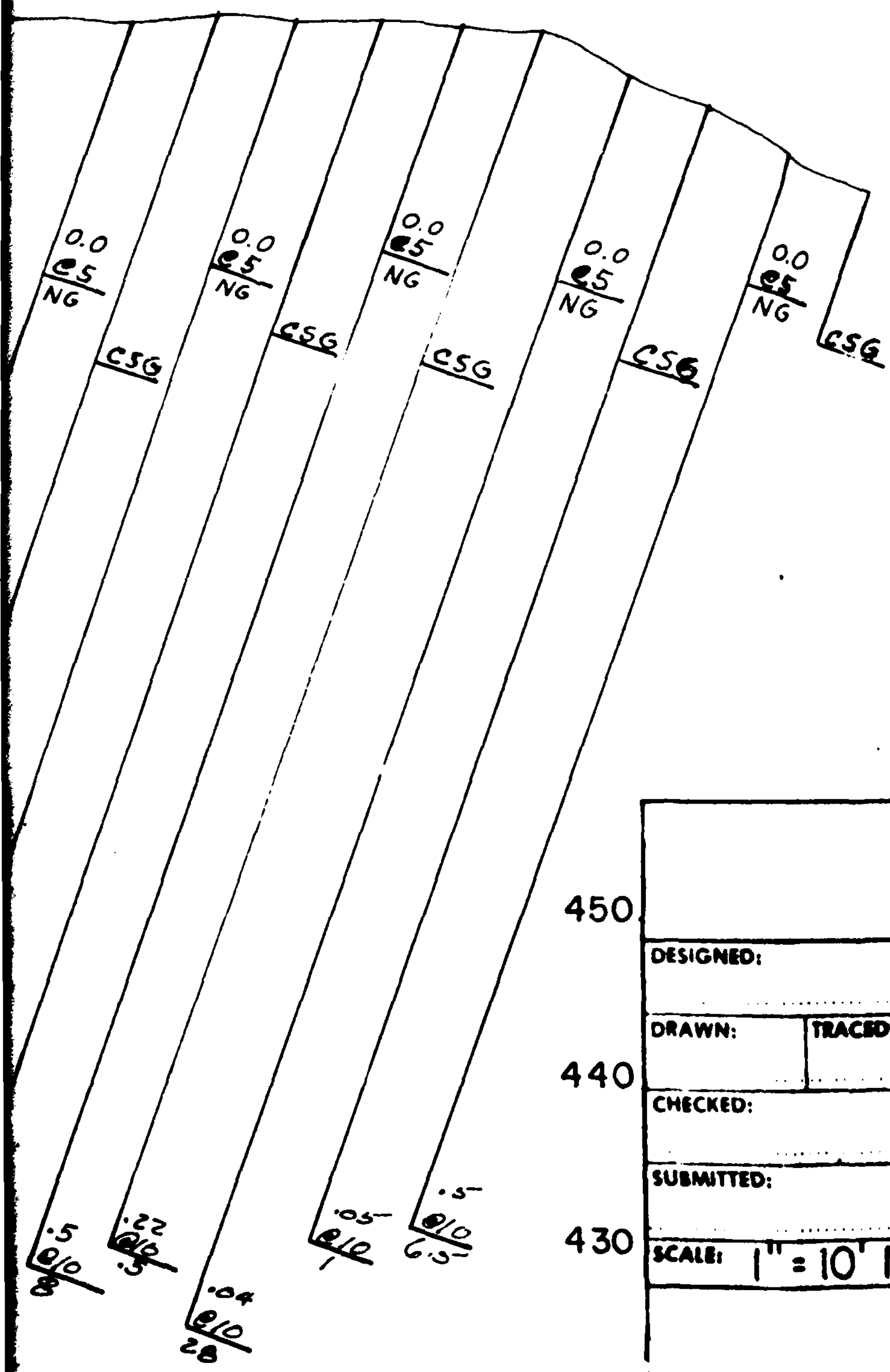

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CORPS OF ENGINEERS
LOUISVILLE, KENTUCKY

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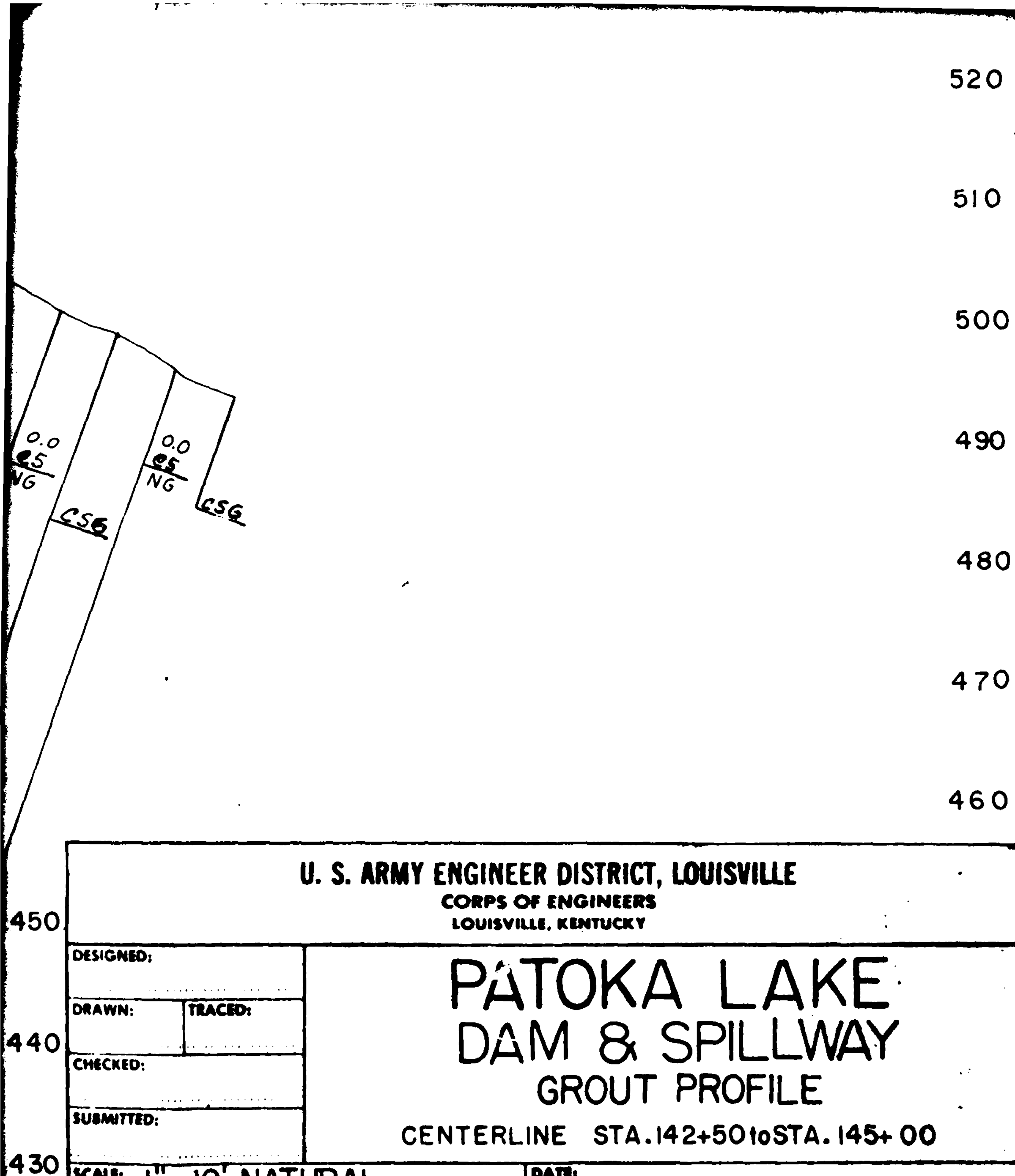

U. S. ARMY ENGINEER DISTRICT, LOUISVILLE
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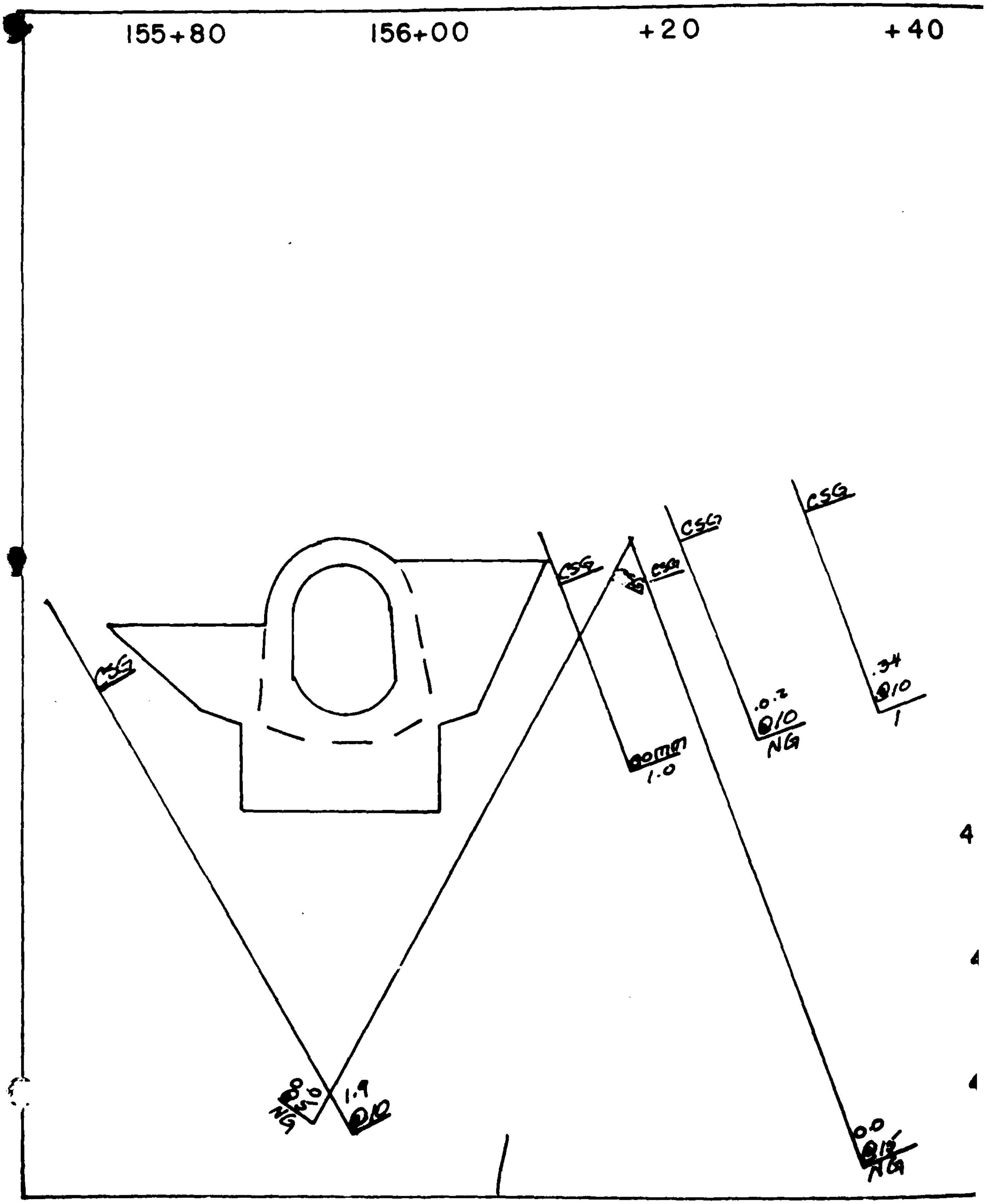

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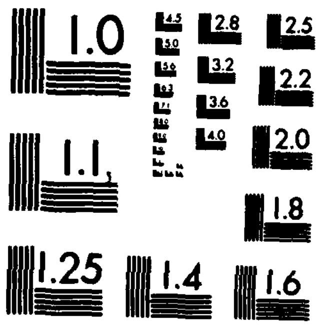

MICROCOPY RESOLUTION TEST CHART
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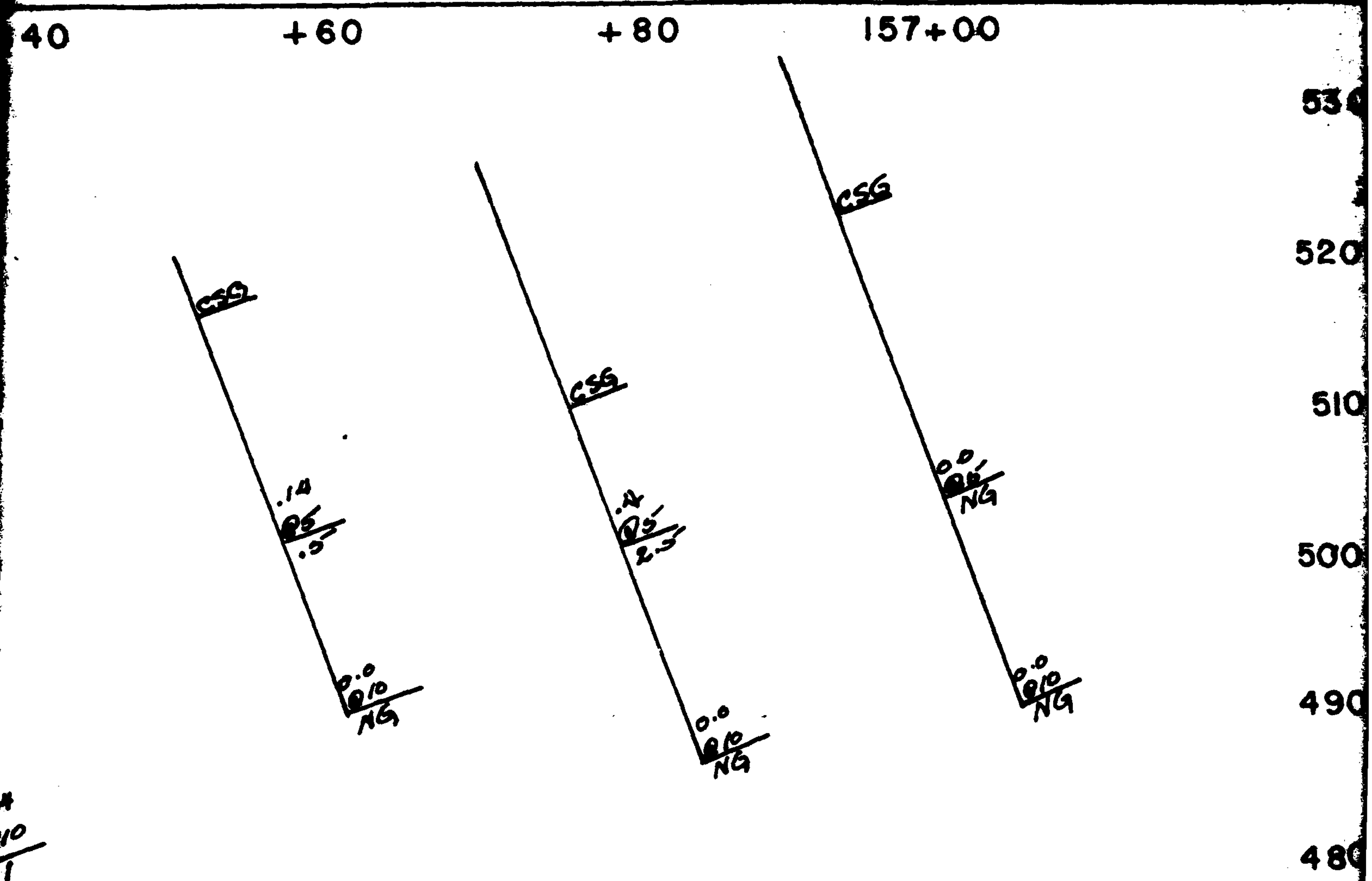

U. S. ARMY ENGINEER DISTRICT, LOUISVILLE
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PATOKA LAKE
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APPENDIX F

CIVILIAN CONSULTANT'S REPORT
PATOKA LAKE FOUNDATION REPORT

SOILS AND ROCK ENGRG.
DAMS AND TUNNELS

Lucas

TEL. 602/977-402

KENNETH S. LANE, P.E.
CONSULTING ENGINEER

10114 KINGSWOOD CIRCLE
SUN CITY, ARIZONA 8535

July 30, 1977

Col. J.N. Ellis
District Engineer
Louisville District - Corps/Engrs
Box 59
Louisville, Ky., 40201

Subject: Consultants Report
Patoka Dam Foundation
Treatment

via Mr. J.M. Kellberg

Dear Col. Ellis:

Attached letter-report (by consultants Kellberg and Lane) covers our Review of Foundation Treatment at Patoka Dam with your staff on July 12-13, 1977.

It represents a joint report, which was developed by our exchanging draft portions and then reviewing the coordinated result. If there are questions, we will be glad to consider them further.

This is being routed first to Mr. Kellberg for addition of his signature and then transmittal to you.

Sincerely,

K.S. Lane

K.S. Lane

Incl: Report, orig.
" , photo-c for Kellberg

July 30, 1977

Col. J.N. Ellis
District Engineer
Louisville District - Corps/Engrs
Box 59
Louisville, Ky., 40201

Subject: Patoka Dam - Consultants Review of Foundation Treatment

Dear Col. Ellis:

This summarizes consultants views as discussed at meeting July 12-13, 1977 to review foundation treatment of Patoka Dam. First day involved a meeting at District Office; second day a visit to the construction site where status was about as follows:

Dam embankment a little above el. 540.
Right abutment grout curtain complete.
Spillway excavation some 10-15 ft. above grade.
Dike cut-off excavation close to complete.

The inclosure lists those participating, also key background data supplied to us for review in advance.

I. Dam Embankment

Main problems are resistance to a possible earthquake and insurance against embankment material piping into the foundation rock.

While perhaps somewhat marginal here, the quake problem has been handled very conservatively by excavating to found the dam on rock - removing the lake deposit of sand and thus eliminating the possibility of its liquefaction by a quake.

The risk of piping the embankment into rock openings (abetted by poor compaction at a rough rock contact) has too often been overlooked on past projects - some examples being the 1960 major leak at Penn Forest Dam, the 1965 partial failure of Fontenelle Dam, and the 1976 failure of Teton Dam. At Patoka the problem was recognized and well handled by defensive design for treating the 3 solutioned limestones (Beech Creek, Golconda and Glen Dean), primarily by: (1) dental concreting cavities, (2) two layer filter between core and downstream rock and (3) concrete facing wall at core contacts over abutment outcrops and each side of valley cut-off trench. Such facing wall has the dual advantage of plugging smaller openings and facilitating good compaction against a smoothed face. Since its construction apparently proved very practical, this facing wall at Patoka may have set a desireable precedent for future designs.

II. Spillway

It appears that top of the Glen Dean Limestone is close to designed grades for the spillway channel. While the spillway is designed as an unpaved channel for rare discharges, we suggest this limestone be regarded as a form of paving which could reduce repairs after a significant discharge. Thus there is some advantage in preserving the limestone, or at least minimizing shattering from blast damage.

Hence we suggest the limestone surface be contoured and spillway details be restudied to take advantage of the limestone. Conceivably this could involve: tolerating a non-planar channel bottom (as not attempting to blast out thin wedges - say 1-3 ft. thick - merely to satisfy a theoretical line); also founding the crest sill on limestone, with perhaps some modest relocation of it.

III. Dike

Key problem is one of cut-off: thru an alluvium filled old channel, eroded to bottom of the Glen Dean Limestone; and thru the adjacent highly solutioned reaches of that limestone. Contract plans contemplate a complete cut-off (backfilled trench) with defensive treatments against piping into the rock similar to those used at the dam.

1. Left Abutment Grouting was tried, but continued high takes suggested unreliable results. Subsequent excavation showed the limestone badly solutioned: pinnacles between widened joints; and cavities partly filled with both residual clay and sand - the latter derived from partial collapse of the overlying friable sandstone. In effect these collapsing cavities represent sink holes in the process of formation which will eventually progress to break-out at the surface. For such "active type" of cavities, grouting is apt to be very undependable since the sand filling would be particularly susceptible to removal by water flows (concentrated by the grout) - thus stimulating the mechanism of cavity enlargement and upward progress of collapse.

Once the condition was revealed, it was decided to extend the cut-off excavation thru the badly solutioned reach to contact reasonably sound limestone. We concur in this decision, and from inspection of the the nearly completed excavation are satisfied that its objective is being attained.

2. Right Abutment Here also the cut-off excavation has been extended, but was stopped where 2 solution-widened joints intersected to form a cave, largely filled with rubble from partial collapse of the overlying sandstone. Since the cave seemed to extend far beyond any reasonble further extension of the excavation, it had been decided to stop here and construct a relatively massive concrete abutment. This was being concreted at time of our visit with generous embedment of pipes for grouting the cave and related cavities.

We consider this a practical solution since the shape of the concrete abutment lends itself to connection with either a triple line grout curtain or with a more positive cut-off diaphram, discussed below.

3. Bottom of Old Channel Here a sand deposit (some 20 ft. maximum thickness) remained to be excavated - mainly because it is carrying water and causing sloughing of the alluvium slopes above. While the correspondence furnished indicates some thought of stopping excavation at top of this sand, we favor following the design intent of extending the cut-off thru this sand to contact the underlying shale; altho its width might be reduced, as to about 8 ft. or even 5 ft.

The current problem is really one of dewatering for construction. Present approach by pumping from gravelled sumps is usually very slow; so if time is pressing, consideration should be given to a more efficient approach - such as a well-point system.

IV. Grouting Program

1. General From our review of grouting procedures and detailed grouting records we believe the grouting program at Patoka Dam has been and is being conducted in accordance with standards reflecting the best state of the art.

No grout curtain based on injection of slurry through small diameter holes can be depended upon to form a completely impermeable diaphragm. Especially is this true in foundations, as at Patoka, consisting of calcareous rocks containing solution cavities which are partially or totally clay-filled. Washing of holes prior to grout injection does not remove all the clay filling and there is no assurance that the remaining clay is confined and rendered immobile by the grout. The best that can be expected from a conventional grout curtain installation is effective reduction of potential underseepage to insure structural stability and to restrain leakage within acceptable limits.

Furthermore there is a growing body of misbehaviour experiences which suggests that in some soluble rocks a grout curtain is not necessarily permanent - some examples being Hales Bar, Wolf Creek and Logan-Martin Dams. Either the solutioning continues, and/or flow concentrations wash out the soil filling of the cavities. At Patoka the sand filling of the cavities in the Glen Dean Limestone would be particularly susceptible to such washing out, as discussed above.

Hence it must be realized that at some time during the life of the project it may be necessary to regrout or otherwise treat portions of the curtain that show evidence of deterioration. Initial curtain alignments should be laid out so that access for maintenance is feasible after impoundment. If this is not possible, some other method of treatment that will insure a positive and permanent cut-off should be employed. With these thoughts in mind we have the following comments on the individual areas at Patoka Dam where grouting has been done or remains to be done.

2. Right Abutment and Right Rim - Sta. 155+00 to Sta. 177+46 Grouting records for the portion of the curtain installed on the right abutment of the dam and extending for approximately 2200 feet along the right rim indicate to our satisfaction that an adequate curtain has been installed. Grout takes were minimal along the entire extent of this curtain, with the exception of one short segment near the crest of the right abutment. In this segment, grouting on closer centers was done until grout acceptance was reduced to within specified limits. Post-impoundment surveillance of this rim area should be made to determine the need for any possible extension of this curtain.

3. Left Abutment - Sta. 140+00 to Sta. 142+00 We concur with the proposed plan to install a triple line grout curtain for at least 200 feet beyond the left end of the dam. Available data indicate the Glen Dean Limestone in this abutment is protected by a shale cap and solutioning appears to be mininal. However, this curtain is advisable to insure against a steep and active hydraulic gradient developing immediately adjacent to the end of the dam.

4. Dam to Spillway - Sta. 142+00 to Sta. 130+50 Throughout this length of the left rim the Glen Dean Limestone is capped by an average thickness of three feet of shale which apparently has inhibited solutional activity in the underlying Glen Dean. Evidence in support of this assumption includes:

A. Fewer cavities encountered in exploratory drill holes, as compared with cavities encountered in the segment between the spillway and the dike.

B. Lower grout takes during the test grouting program, compared with high takes in the area to the left of the spillway.

C. Higher natural groundwater table elevations in this area.

D. No indications of water or dye connections from borings in this area to the major spring approximately 2500 feet northwest.

E. No surface evidence of collapse structures in the overlying Mansfield Sandstone.

Although it is possible that significant leakage could develop through this portion of the rim after impoundment, we do not consider the probability nearly as great as leakage through the segment between the spillway and the dike.

We recommend the following treatment for this area. First, since grout nipples have been installed 10 feet on centers along the proposed grout line, these holes should be grouted. If there are localized areas with significant takes, grouting should be done on reduced centers at these localities to obtain closure. Second, as a reinforcement for the minimal single line curtain, we recommend that the outcrop of the Glen Dean Limestone along the adjacent reservoir slope be blanketed with waste material - as a minimum up to a little above the normal pool elevation at 536. We believe the combination of the single line curtain and the protective upstream blanket is the most economical method for pre-impoundment treatment of this area. Future surveillance after impoundment will determine if or when additional treatment is required.

5. Spillway to Dike - Sta. 130+50 to Sta. 117+45 From our observations in the spillway excavation and at the right abutment of the dike and from logs of holes drilled between the spillway and the dike it is apparent that the protective shale cap between the Glen Dean Limestone and the overlying Mansfield Sandstone is absent in this area as the result either of non-deposition or of pre-Mansfield erosion. Consequently the Glen Dean Limestone is severely solutioned. Evidence for this includes:

A. The numerous cavities and weathered zones encountered in cores from holes drilled in this area.

B. Collapse structures in the overlying Mansfield Sandstone. Some twelve of these have been located in the spillway excavation and several more in the dike excavation.

C. Extremely high grout takes throughout this area during the test grouting program.

D. Direct dye connections from holes drilled in the area to the large spring approximately 2500 feet northwest and downstream.

E. Low groundwater table prior to construction activity.

The Boundary between the highly solutioned Glen Dean and the less severely weathered Glen Dean seems to be a line trending slightly west of north crossing the grout line at the right side of the spillway excavation. Apparently this marks the limit of protection afforded the Glen Dean by the shale cap.

It is our considered opinion that any attempt to install an effective conventional grout curtain between the right end of the dike and right side of the spillway excavation would be utterly futile. Vast amounts of slurry could be injected at high cost over an unpredictable area during an indeterminate period of time with no assurance of a permanently reliable cutoff being obtained. We concur with a propoasl such as, or similar to, the one suggested by your staff on Page 3 of the ORD memorandum of

6 July 1977 to install a positive cutoff wall of concrete through the Glen Dean Limestone from the right end of the dike to the right side of the spillway. Such a cutoff should be installed from a bench excavated on the top of the Glen Dean. The exact method of trench excavation to be used, whether a pre-split and blasted narrow slot or intersecting large diameter holes, for example, should be determined by economic evaluation and contractural capabilities.

If equipment and crew are reasonably available, the flame jet could be another possibility (as being tried for a 60 ft. deep trench at Bailey Dam) provided abrasive is mixed with the flame - a technique developed by James Browning (Browning Engr. Co., Hanover, N.H.) to make the flame jet efficient in cutting softer rocks and those composed of essentially a single mineral (limestone, sandstone, etc.).

Upon completion of the cutoff wall the excavation area should be refilled with impervious and random waste material to provide an additional protective blanket. Extending the refill blanket up to about spillway crest (el. 548) also deserves consideration to minimize the possibility of sustained flow thru the Mansfield Sandstone reaching the Glen Dean Limestone to reactivate cavity enlargement downstream of the cut-off wall.

We consider this area to have the highest post-impoundment leakage potential of any area of the project and believe it deserves a maximum treatment effort.

V. Observation Program

Instrumentation includes: surface movement points plus 2 accelerographs for earthquake monitoring; and for the leakage problem, an array of piezometers (in Golconda and Glen Dean Limestones) plus a wier at existing cave spring 1/2 mile downstream. This seems initially adequate, with the probability that early observations will show where a few more piezometers would be desireable.

Since the leakage problem is very real and the adequacy of its various treatments can be determined only by prolonged observations during operation, we suggest a plan be developed for deliberately testing out different reservoir stages in the early years of project operation. Conceivably this could be along the following lines:

1. Low stage - holding pool above Golconda Limestone to test its grout curtain.

2. Normal operating stage - permanent pool at el. 536, which covers only lower part of the Glen Dean Limestone.

3. High operating stage - during non-flood season holding highest practical pool to cover most of the Glen Dean Limestone and thus better test its grouting. Around el. 542-44 could be a desireable target if such is attainable without undue risk to flood-control capability.

K. S. Lane

K.S. Lane, Consulting Engineer

John M Kellberg

J.M. Kellberg, Consulting Geologist

1 incl.

PATOKA LAKE, INDIANA

Meeting of 12 July, 1977

*Col James N. Ellis	District Engineer
Kenneth S. Lane	Board Member
John M. Kellberg	Board Member
Wm. Leegan	Chief, Engineering Division
Jack Kiper	Chief, Construction Division
Steve Markwell	Chief, S & I Br.
Albert Harrison	Chief, Foundations & Materials Br.
Eugene J. Miller	Chief, Soils Design Sec.

* Part time

Key documents supplied for advance review

Embankment Design Memo No. 8 & Supplement No. 1
Plans & Specs.
Reports of numerous construction inspections over period
27 June 1975 thru 30 June 1977

APPENDIX G

GOVERNMENT PERSONNEL ROSTER
TO PATOKA LAKE FOUNDATION REPORT

Roster of permanently assigned personnel during Construction of Dam, Dike and Spillway:

Name	Position	Length of Tour
Meetze, O. L.	Resident Engineer	Jan 1975 - Dec 1978
Stuart, Charles T.	Geologist, Embankment Inspector	Jul 1975 - Aug 1977
Frick, Mona L.	Clerk	Mar 1977 - Aug 1979
Knight, Carolyn S.	Clerk, Engineering Technician	Jun 1975 - Nov 1980
Hopper, Billy G.	Office Engineer	Mar 1975 - Dec 1977
Fitzgerald, Gary V.	Assistant Resident Engineer, Resident Engineer	Jul 1977 - Dec 1978 Dec 1978 - Oct 1980
Markwell, Richard	Office Engineer	Dec 1977 - 1981
Nation, Darrell J.	Field Engineer, Office Engineer	Oct 1975 - Feb 1980
Batte, William F.	Materials Technician	Jan 1975 - Oct 1980
Cassidy, Howard W.	Inspector	Jan 1975 - Jul 1980
Pagioili, John	Embankment Inspector	Sep 1975 - Oct 1978
Rainey, Anthony D.	Geologist	Nov 1977 - Aug 1978
Bartlett, S. R.	Project Geologist, Embankment Specialist, Geotechnical Representative During Filling, Foundation Report Writer	Apr 1975 - Aug 1979

APPENDIX H

BIBLIOGRAPHY TO PATOKA LAKE
FOUNDATION REPORT

BIBLIOGRAPHY OF RELATED REPORTS

Due to the complexity of this project, many unique and unusual methods of correcting the foundation deficiencies were utilized. As a result of the unique methods of treatments, many papers and reports were written about Patoka Lake. As a guide to further foundation evaluation, a bibliography of the various extra reports, articles and papers concerning the foundation treatment has been included. The preconstruction design memorandums and early state geological survey reports have not been included in this bibliography. This bibliography may not be complete since further articles may have been published after this report was finalized.

1. Cooper, Stafford S. and Bieganousky, Wayne A., "Geophysical Survey of Cavernous Areas, Patoka Dam, Indiana," Miscellaneous Paper S-78-1, January 1978, Waterways Experiment Station, Vicksburg, Mississippi. Report of geophysical methods to delineate cavities and solution features in Glen Dean Limestone between the dike and spillway.

2. Couch, Frank B. and Ressi de Cervia, Auturo L., "Seepage Cutoff Wall Installed Through Dam is Construction First." Civil Engineering - ASCE, January 1979, Page 62-66. Included in this article is a short insert on the cutoff trench at Patoka Lake by Claude A. Fetzer.

3. Fetzer, Claude A., "How San Fernando Earthquake Saved Patoka Dam," World Water, December 1979, Page 20-22.

4. Fetzer, Claude A., "Patoka Dam - Foundation Treatment," 13th Congress on Large Dams (ICOLD), New DeLhi, India, 29 November - 2 December 1979. Presentation of a discussion on question number 48 at this meeting.

5. Fetzer, Claude A., "Patoka Dam Tested," USCOLD Newsletter, November 1979, Page 5-9. A short synopsis of design, construction and impoundment.

6. Gray, Henry B., "Geology of the Upper Patoka Drainage Basin," Special Report Number 2, 1963, Indiana Department of Natural Resources, Geological Survey, Bloomington, Indiana. A report of the first site investigations performed at the damsite.

7. Gray, Henry B., "Glacial Lake Deposits in Southern Indiana - Engineering Problems and Land Use," Department of Natural Resources, Geological Survey Report of Progress 30, Bloomington, Indiana, 1971. Brief discussion of glacial lake deposit occurrences and engineering problems.

8. Kelley, Benjamin I., "Foundation Treatment Patoka Lake Project, Indiana." A paper presented at the Corps of Engineers Geotechnical Conference in Portland, Oregon on 19 - 22 October 1976. A brief description of foundation design and treatment to that date.

9. Kelley, Benjamin I. and Markwell, Steven D., "Seepage Control Measures at Patoka Dam, Indaina." ASCE Reprint 3455, Chicago Convention Exposition, 16 - 20 October 1978. Brief synopsis of the foundation treatment for this project.

10. Krinitzky, "Geological and Seismological Factors for Design Earthquakes, Patoka Damsite, Indiana." Miscellaneous Paper S-72-42, December 1972, Waterways Experiment Station, Vicksburg, Mississippi. An in-depth analysis of seismological history of the damsite.

11. Marcuson, W. F. and Gilbert, P. A., "Earthquake Liquefaction Potential at Patoka Dam, Indiana." Miscellaneous Paper S-72-41, December 1972, Waterways Experiment Station, Vicksburg, Mississippi. An in-depth analysis of the liquefaction potential of the sand-silt foundation of the original design.

12. Marcuson, W. F., "One-Dimensional Wave-Propagation Analysis, Patoka Dam, Indiana," Miscellaneous Paper S-74-26, November 1972, Waterways Experiment Station, Vicksburg, Mississippi. Report on results of triaxial shear tests on undisturbed samples.